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AVAILABLE HEAT SINKS FOR PROTECTED UNDERGROUND INSTALLATIONS

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INTRODUCTION

All heat released within an underground installation during an emergency period must be absorbed by some suitable means to maintain tolerable conditions. Some of the heat inputs include heat from electronic equipment, lights, electric motors, personnel, outside air, electric generating equipment, and air-conditioning equipment. Present indications are that rates of heat gain from these sources for military underground installations range from 4 to 15 million Btu/hr.

In view of the possibility of being cut off from outside facilities for heat disposal, an adequate heat sink is required within a military underground installation for use during an emergency period. As used herein, a heat sink is defined as a body of great thermal capacity relative to the rate at which heat is to be furnished to it. The simplest type of heat sink for heat rejected from water-cooled equipment such as a Diesel engine, or from the condensers of air-conditioning systems, is a water-filled underground reservoir initially at a suitably low temperature. Also, if the initial temperature of the rock surrounding an underground installation is suitably low, it has an enormous capacity for absorbing heat, although the rate at which it can accept heat is limited and decreases with time. However, if the entire facility is operated under normal or non-emergency temperature conditions for a considerable prior time, the rock surrounding an installation will be warmed and its effective capacity for heat absorption may be small at the time of an emergency. A proposed special air-conditioning arrangement designed to reserve the heat-sink capacity of the surrounding rock will be outlined later.

Since 1950, the National Bureau of Standards, under a working agreement with the Office of the Chief of Engineers, Department of the Army, has been concerned with various problems dealing with heating and air conditioning of underground installations. The ultimate objective of the undertaking was to prepare a design manual on this subject.* The work included a survey, experimental studies in an actual underground chamber and other installations, and theoretical considerations of the various heat transfer problems.

EXPERIMENTAL INVESTIGATIONS

The experimental and analytical problems completed in this study are briefly enumerated as follows:

1. Heat absorption by rock surrounding an underground installation. The main purpose of this investigation was to derive data on the rate of heat absorption by the rock to determine the net heating or cooling loads for an installation.

2. Heat flow from or to the rock surrounding outside air intake tunnels or shafts. The purpose was to determine the reduction, if any, of the heating or cooling load of an installation by the warming of air in the winter or the cooling of air in summer by the surrounding rock of the tunnel.

3. Heat absorption by rock surrounding an underground water reservoir used as a heat sink. The underground reservoir to be used for the disposal of waste heat during an emergency period was studied because of the evident vulnerability of outside cooling water services such as water mains and cooling towers.

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*The manual, EM 1110-345-451, entitled Heating and Air Conditioning of Underground Installations, is available from the Office of the Chief of Engineers only to Government agencies.

4. <u>Iced underground reservoirs</u>. Methods of forming or distributing ice along the length of an underground water reservoir were investigated mathematically and experimentally.

Figure 1 shows an underground chamber used to determine rates of heat transfer to the surrounding rock. The chamber was 100 by 35 by 10 ft, and was located 230 ft below the surface. For illumination purposes, rock walls were painted white with a cement water paint. Heat was supplied electrically to the recirculated air system and numerous thermocouples for measuring temperatures were placed in the air and on and in the surrounding rock.

Figure 2 shows the actual temperatures and heat input rates per square foot of projected rock surface area during a heating experiment in the underground chamber, and the values calculated analytically on the assumption that the actual chamber was a cylinder or a sphere of equal internal area. In this experiment, the chamber, initially at the rock temperature of 54°F, was heated for six days at a constant rate of heat input, and thereafter heat was introduced as needed to maintain a steady air temperature of 75°F. Means were established to decide whether a sphere or a cylinder is the best approximation to an actual chamber, and to correct the heat flux calculated for the assumed shape to yield that for the actual chamber.

Figure 3 shows the temperature and heat input rate history of a 1.5 million gallon reservoir during a test with an approximately steady heat input rate averaging 6.1 million Btu/hr. The reservoir, dammed at one end, was 34 ft wide, 29 ft high, and 230 ft long, filled with water to a depth of about 25 ft, and was initially at 50°F. Water was taken from the bottom of the reservoir and passed through heat exchangers where it absorbed heat, and the warmed water was returned to the reservoir through spray nozzles to bathe the rock surface exposed above the water level. The warmed water, due to its lower density, tended to stay on top and to descend to lower levels only as colder water was taken from the bottom.

Stratification accounts for the increasing lag of temperature rise of the water at increasing depths. Little rise occurred in the outgoing water temperature the first 52 hours. Then, the outgoing water temperature started to increase rapidly, and a corresponding rise therefore occurred in the temperature of the incoming water. Stratification again accounts for the subsequent period of slow outgoing water temperature rise, which is also reflected in the incoming water temperature rise, although these rates are modified by the changes that occurred in water flow rates. The test was terminated at 117 hours, when the average temperature of the water in the reservoir approximated 101°F.

Average water temperatures taken from Fig. 3 are shown by the circles in Fig. 4. Curve A is a straight line showing what the water temperature history would have been if none of the heat had been absorbed by the surrounding rock. Curve B shows the analytically-predicted water temperature history. The close agreement of the average water temperatures with the mathematical prediction is gratifying, considering the limited number of samples of the rock available for laboratory determination of their thermal properties.

By means of a heat balance, it was shown that at 117 hours, the surrounding rock had absorbed about 11 per cent of the total heat added to the reservoir, thus increasing its useful life by about 13 hours. It can be shown that for longer time durations, greater percentages of the heat added to the reservoir would be stored in the rock.

ENGINEERING AND MATHEMATICAL CONSIDERATIONS

Circulation of outdoor air into and out of an underground installation to dispose of waste heat during an emergency is possible but not promising. Among the problems involved are the large size of airways, the power required to move the air, and the possibilities of blast damage to the airway or interior equipment and of contamination of the air.

However, air inside the installation may serve well as a medium for transferring heat to the exposed rock surfaces.

For transporting heat from equipment heat exchangers to a heat sink such as a water reservoir, water is, of course, the obvious and best medium. During an emergency, the water in the reservoir is pumped to the heat exchangers and returned at a higher temperature. A portion of the heat added to the reservoir is conducted into the surrounding rock.

If the heat input rate to the reservoir is constant, the thermal behavior of the reservoir can be determined by mathematical analysis, which therefore provides a basis for calculating reservoir size. Solutions obtained by mathematical analysis, assuming the reservoir can be approximated by a cylinder of equivalent perimeter, have been evaluated by a digital computer. The results are expressed in the form of plotted functions of dimensionless variables involving time, the thermal properties of the rock and water, and the equivalent diameter of the reservoir. The importance of being able to calculate reservoir sizes accurately may not be too evident until it is realized that underground reservoirs with capacities up to 7 million gallons of water have been excavated for this purpose, and that the amount of excavation may be 30 per cent or more of the total excavation necessary.

Table 1 shows reservoir sizes computed for a hypothetical problem for emergency durations of 6, 10, and 14 days. Also shown is the percentage of heat stored in the rock at the end of each duration. These percentages represent heat input rates of 2 million Btu/hr for 1.2, 2.4, and 3.9 days in the three cases, respectively. These cases show that to design reservoirs where heat absorption by the surrounding rock is neglected can give considerably larger reservoir sizes than necessary.

When the water in the reservoir reaches its maximum allowable temperature, it can be passed through heat exchangers and wasted to the outside, to extend the period of operation under emergency conditions. For the design of reservoirs, consideration must be given to the fact that condensers of refrigeration systems used for air conditioning generally do not allow entering water temperatures higher than 100^{o}F. Temperatures above this may be damaging to the equipment, due to high compressor head pressures. On the other hand, heat exchangers of prime movers such as Diesel engines may allow water temperatures up to 160^{o}F or higher. Because of the difference in the allowable entering water temperatures, separate reservoir systems should be designed to accept heat from the two separate heat sources, although in some instances water being wasted from the air-conditioning reservoir and system can be passed also through the prime mover coolers on its way outside.

An underground reservoir generally is open on one end only, with a suitable dam, and should have provision for taking water from its lowest elevation and returning warmed water at the top. A suitable barrier should be placed above the dam so that water vapor from the warmed water will not be transferred to other parts of the installation. Reservoirs should be placed at the lowest elevation in relation to the the rest of the installation to prevent possible flooding.

A few engineering considerations will be useful and pertinent in relation to heat loads, heat sinks, and their interaction. For an average air-conditioning system, the coefficient of performance is about five. That is, to absorb one Btu, the required motor energy input to the compressors must be about 0.2 Btu. The mechanical and electrical efficiency of the power drive system from the prime mover generator to the compressor is on the order of 80 per cent. Therefore, the energy required is about 0.25 Btu, but the heat losses of the drive increase the air-conditioning load by about 0.05 Btu. Thus, to effect one Btu of net cooling, 1.05 x 0.25 or 0.263 Btu must be used, and the heat that must be absorbed by the air-conditioning reservoir is 1.26 Btu. Further, since a good prime mover under load rejects to its cooling system an amount of heat approximately equal to its shaft output energy, the prime mover waste heat reservoir has to absorb about 0.25 Btu for each Btu of net cooling effect of the air-conditioning system. These approximate figures emphasize the great importance of restricting air-conditioning loads that require compressor operation during periods of emergency operation, either to reduce the heat-sink capacity required, or to stretch out the thermal stamina of a given heat sink installation.

ARTIFICIAL COOLING OF RESERVOIR HEAT SINKS

Cooling of the water in a reservoir by refrigeration during a non-emergency condition will maintain the water and surrounding rock at a temperature below its original value. This will allow more heat-absorbing capacity for the emergency period or a reduction in the designed size of the reservoir. If and as long as this water is maintained at a temperature below that necessary for accomplishing air conditioning (approximately 50°F), it may be used directly in air-conditioning coils, and power would not be required to operate the refrigerating compressors for air conditioning during this time.

A practical ultimate in heat-storage capacity is to maintain a reservoir filled with an ice and water mixture. The latent heat of fusion of ice gives, for each pound of ice, the equivalent of about 144°F temperature rise of water. Considerable additional heat-absorbing capacity is provided by the nearby surrounding rock, since it also has been cooled down to about the freezing temperature of water. The design for such a reservoir demands consideration of the method of distributing ice along the length of a reservoir. A practical method appears to be to use ice in small pieces, distributed longitudinally in the reservoir by a helical screw located a few feet above the water level.

Table 2, a hypothetical problem involving reservoirs intended for the waste heat from the air-conditioning system, indicates the advantages of reduced initial reservoir temperatures. Average deep-earth temperatures in most of the United States are between 42° and 72°F, depending on location; in the first two examples, natural initial reservoir temperatures of 65° and 52°F are assumed. The third column shows the size for a reservoir maintained at 40°F, either naturally or by means of refrigeration during non-emergency conditions. A significant point in this case is that during an emergency, waste heat from the refrigeration compressors is not included during the first 10° temperature rise of the water. Also, the power-generating equipment does not have to provide the power to drive the refrigeration compressors during this period. The fourth column shows the size for a reservoir maintained with a 40 per cent ice-water mixture by suitable ice-making equipment.

<u>UTILIZATION OF ROCK SURROUNDING AN INSTALLATION AS A HEAT SINK</u>

For a number of reasons (including need for a roof to protect against dripping water or spalling rock, subdivision of space, privacy, and acoustics), underground installations of the kind considered generally are designed to have an "interior structure" or enclosure built within each of the rough excavated chambers. The few feet of space between the interior structure and the surrounding rock are referred to as the "annular space." The figure of Table 3 illustrates a cross-section of a chamber with its internal structure.

Assume that the internal structure is air conditioned during normal or non-emergency periods by the "cold plenum" system shown schematically in Table 3, with air (except for that exhausted as vitiated air) being discharged from the internal structure into the annular space through the cooling coil at the natural undisturbed rock temperature, T. The cool air from the annular space would be admitted into the internal structure at the top through diffusers, at the rate necessary to maintain the desired interior conditions. The non-emergency air-conditioning load would be equal to the heat liberated within the internal structure. Much more importantly, however, the large mass of surrounding rock would be kept at the relatively low temperature, T, and during an emergency would be available as a heat sink for the internal structure.

Referring to Table 3, the rock is capable of absorbing heat for 20 days at the rate of 6.0 Btu/hr per square foot of projected surface area, for an annular space air temperature rise to 75°F from an initial temperature of 52°F (i.e., 23°F). For a typical installation, the ratio of annular space rock surface area to the floor area of the internal structure is about four to one. Therefore, for the 20-day case considered, the rock is capable of absorbing for 20 days an interior structure heat load equivalent to about 24 Btu/hr per square foot of floor area, or 7 watts/sq ft. Heat liberation in many spaces in an underground installation, such as dormitories or offices, may not exceed this figure, and such spaces could be cooled adequately during an emergency by the heat-absorptive capacity of the annular space rock alone, without mechanical cooling during the emergency, and thus without imposing a load on the reservoir heat sink capacity of the installation. For spaces with a greater rate of internal heat liberation, mechanical cooling during an emergency would be needed, but the cool surrounding rock would serve to absorb a fraction of the heat load.

For an underground installation in a region where natural deep-earth temperatures exceed 50° or 55°F, it may be desirable to cool the surrounding rock during the non-emergency period to put in store a greater heat sink capacity for use during an emergency. This can readily be done by discharging the air into the annular space at the desired low temperature. The cooling of the rock would, of course, entail a greater cooling load for the cooling coil and refrigerating equipment during the non-emergency period.

With the cold plenum arrangement, a moderate degree of insulation of the interior structure is desirable to reduce excessive cooling of the interior by heat transmission through the structure. For this purpose, the thermal transmittance of the walls, floor, and roof should not exceed 0.2 Btu/hr ft^2 °F for an internal structure with an internal heat load of approximately 7 watts/sq ft; for a considerably greater heat load, insulation is not needed. The structure does not require a vapor barrier, since the air dewpoint is substantially the same on both sides.

Use of the annular space as a cold plenum has the great advantage that it preserves and maintains--as an indestructible capital--the great heat-absorbing capacity of the surrounding rock for use in increasing the thermal stamina of an underground installation during an emergency period.

FIGURES

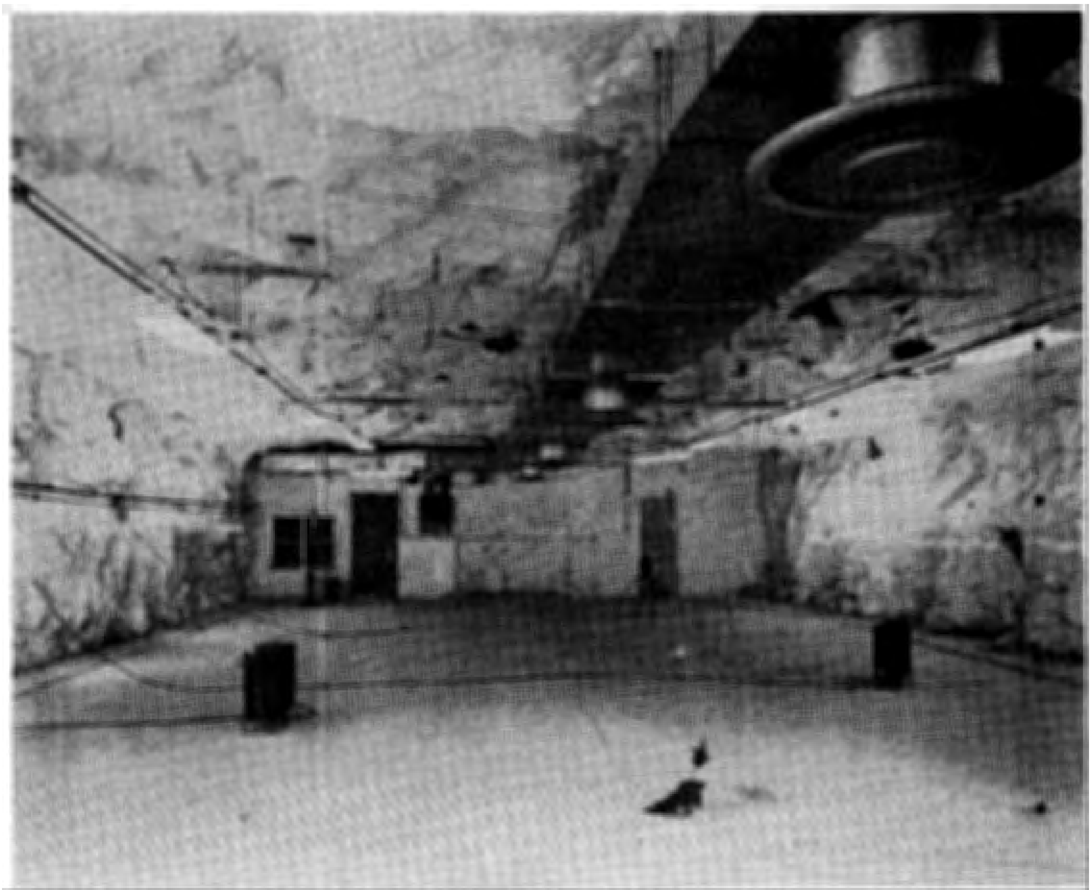
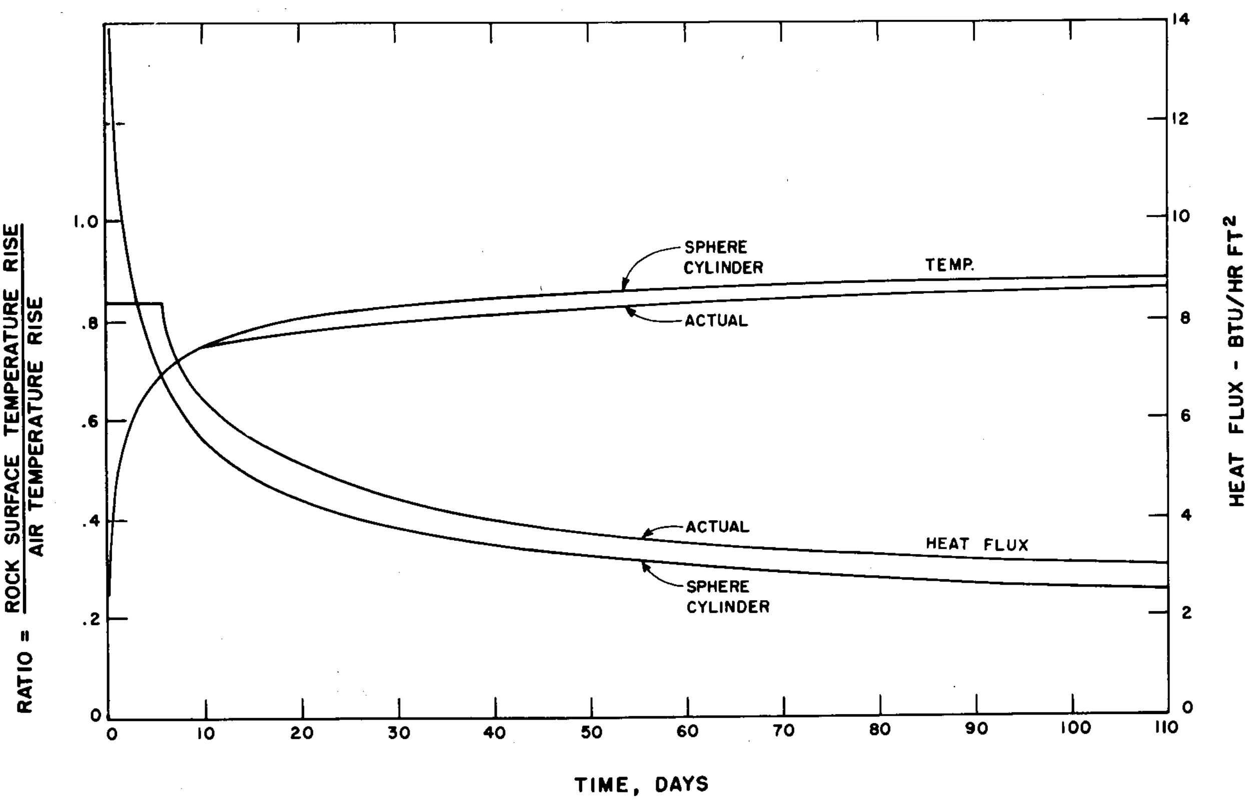


Fig. 1


RATIO = ROCK SURFACE TEMPERATURE RISE / AIR TEMPERATURE RISE
HEAT FLUX - BTU/HR FT2
TIME, DAYS
SPHERE
CYLINDER
TEMP.
ACTUAL
ACTUAL
HEAT FLUX
SPHERE
CYLINDER
0
.2
.4
.6
.8
1.0
0
2
4
6
8
10
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14
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100
110

Fig. 2

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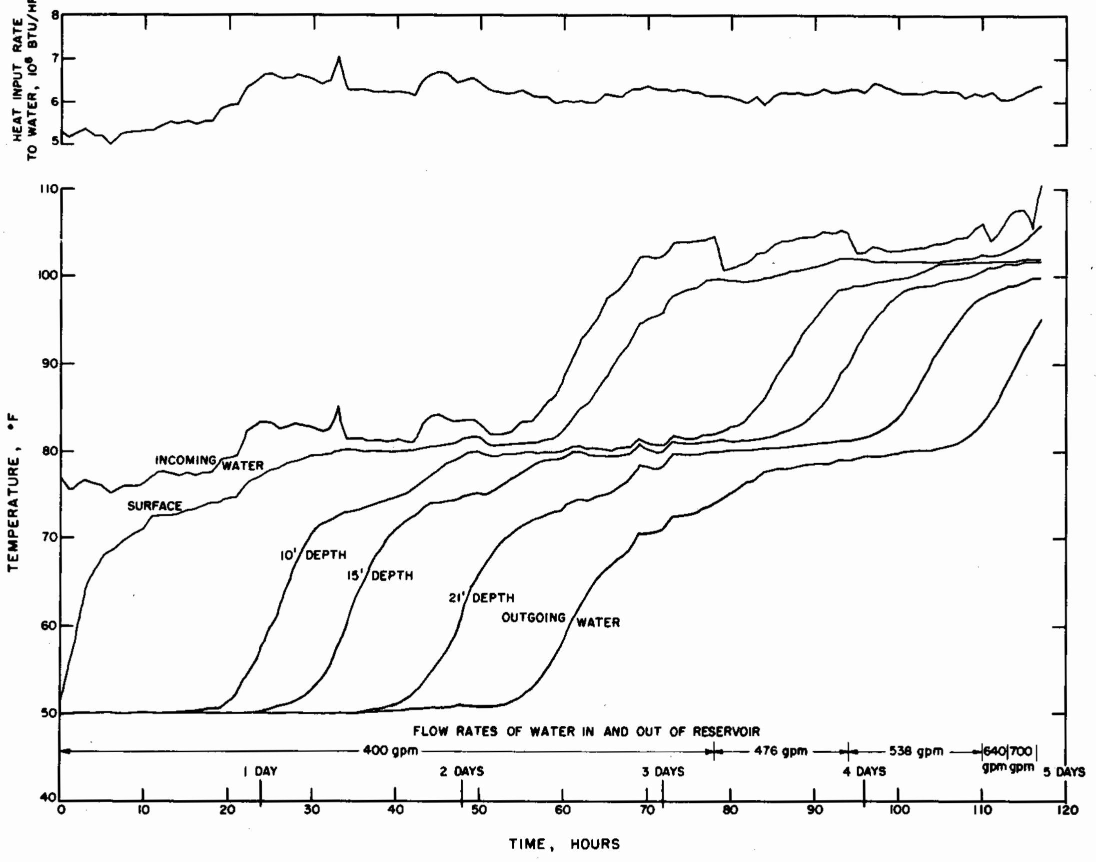


Fig. 3

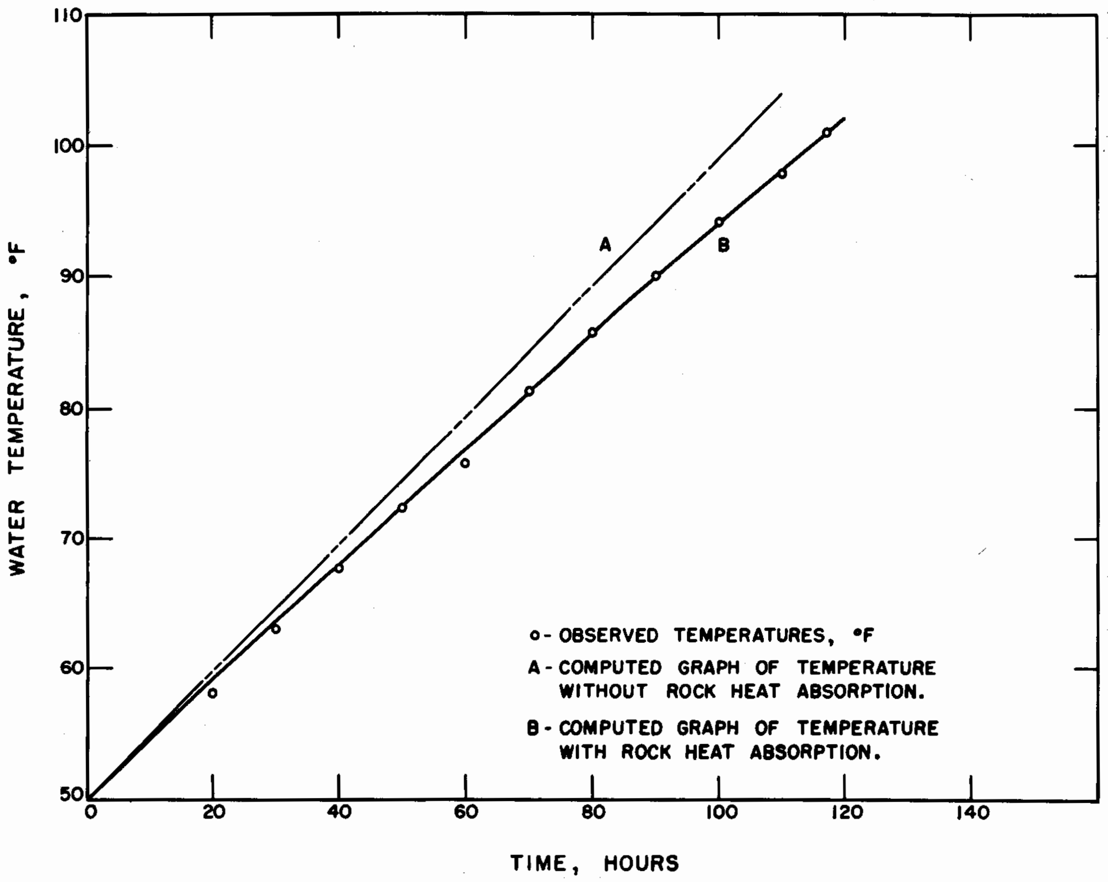


Fig. 4

TABLE 1

Required Reservoir Capacities with Heat Input Rate of Two Million Btu/hr, Reservoir Cross-Section of 20 ft by 20 ft and an Average Water Temperature Rise of 48°F

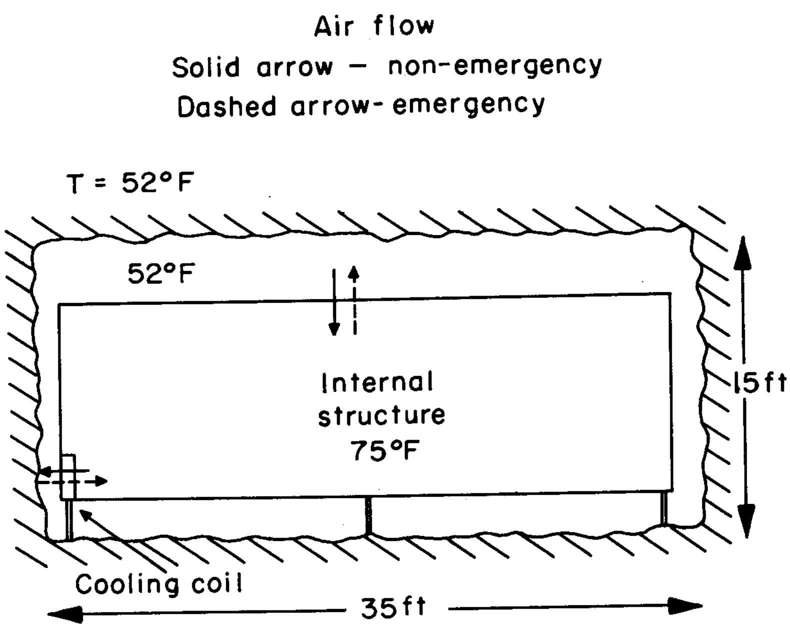
	Time of Emergency Period, Days		
	6	10	14
Length of Reservoir, ft	192	302	405
Water Volume			
Cubic Ft	76,800	120,800	162,000
Gallon	574,500	903,600	1,211,800
Heat in Rock, %	20.1	24.1	27.8

TABLE 2

Hypothetical Problem Considering Air Conditioning Heat Gains Only - Reservoir sizes necessary for 30-day emergency period

Air conditioning load	$= 2 \times 10^6$ Btu/hr
Power required to drive refrigeration	$= 0.4 \times 10^6$ Btu/hr
Total waste heat	$= 2.4 \times 10^6$ Btu/hr
Highest permissible water temperature	= 100F
Cross-section of reservoir	= 20' high x 20' wide

			Reservoir water used in cooling coils until its temp. is 50F. Refrigeration equipment not used during this period	
Initial Water Temp., °F	65	52	40	32(40% ice)
Length, ft	1,280	931	730	420
Volume				
Ft^3	510,700	372,400	292,000	168,000
Gal.	3,820,000	2,786,000	2,184,000	1,256,000



Cross—section of tunnel-shaped chamber in rock

Emergency time duration, days	Heat flux btu/hr - sq ft of projected rock surface area
15	6.60
20	6.00
25	5.54
30	5.20

Table 3—Average heat flux to rock in raising cold plenum air temperature from 52°F to 75°F

UTILITY PROBLEMS IN SUBMARINES AND THEIR RELATION TO UNDERGROUND CONSTRUCTION

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The design and construction of a submarine which operates hundreds of feet beneath the ocean's surface, completely isolated from the earth's atmosphere, presents many problems. Some of these are similar to those which would be encountered in the design and construction of a land-based sub-surface facility which is to be completely isolated from the earth's atmosphere.

The submarine is a large but very compact ship. During World War II the gross internal volume of the submarine was approximately 50,000 cu ft. Today our attack-type submarines have nearly twice that internal volume. Submarines such as those now being constructed to carry and fire the POLARIS missile are approximately three times as large or have a volume of about 150,000 cu ft. Submarines today go deeper, faster, farther, and thanks to nuclear power, can stay submerged many times longer than those in the service in World War II. The heart of the modern submarine is the nuclear propulsion plant consisting of the nuclear reactor and its associated steam generators, steam turbines for propulsion, and turbo-generator sets for generating electrical power.

In addition to the complexity brought about by nuclear power, submarines today have a very large quantity of complicated electronics equipments. Sonar equipments provide the eyes and ears for the nuclear powered submarine that remains completely submerged nearly all the time while at sea. As weapons have become more powerful, so also the equipments needed to control them have become more complex.

One problem in submarine design has remained unchanged, however. The volume of the submarine must be adequate to contain the equipment to be installed but it cannot be oversized without paying a steep penalty in loss of speed, loss of endurance and increased cost. When submerged the submarine must be able to obtain neutral buoyancy. That is, the weight of the ship and all its equipment must equal the weight of sea water displaced by the hull envelope. Since the density of sea water is relatively constant at some 62-64 lb/cu ft, the submarine must be very compact. We cannot afford any waste space. Many spaces must be used for several purposes; for example, the torpedo room is also a bunkroom for part of the crew, the mess hall is also the crew's recreation center.

Since the submarine, in its natural environment, is surrounded by sea water we do have the advantage of an excellent heat sink, but this same ocean brings many difficulties to the design and construction problem. The submarine hull and all the penetrations through the hull must be able to withstand the pressure of the external sea, which increases approximately 1 lb/in^2 with each 2 ft increment in submerged depth. This means particular attention to structural design and welding, and to the penetrations for electrical cables, antenna coaxial cables, to the design of access hatches, penetrating masts and rotating shafts. The corrosive nature of seawater is another feature which forces the design to use special materials and gives the submarine operator real maintenance problems during the life of the ship.

The modern submarine represents a series of engineering and military compromises. Speed, maneuvering, operating depth, quietness (to avoid detection by the enemy and to improve the performance of the submarine's own sonar) weapon effectiveness, habitability, reliability, and cost are all factors that have to be weighed and the best possible compromise reached in order to obtain a balanced design. Once the overall boundaries of the submarine have been established by these considerations, the detailed design follows.

When planning the utilities of a submarine, the designer must consider factors such as reliability, maintenance requirements, weight, space, heat loss, noise (both airborne and that which is transmitted through foundations and structure to the water), stand-by or emergency provisions and, of course, cost--both initial and ultimate. The final choice is, in many instances, the best compromise that can be reached and not a choice which is optimum all the way around. It is necessary, also, to use judgment in providing a system which will meet practical operating conditions and be adequate for emergencies but not overdesigned to fulfill unrealistic situations.

First, let's look briefly at the utility systems on submarines and then we can discuss the design approach to one of these in some detail.

Electrical Systems. The electrical output of the turbo generators on nuclear-powered submarines is 440-volt, 3-phase, 60-cycle power. This is used directly for all large motors such as the driving motors for pumps and compressors and is transformed to lower voltage or converted through motor generators to 400-cycle or d.c. power for auxiliary purposes. Auxiliary power is fed to distribution panels in selected locations throughout the ship. 115-volt a.c. is used for fluorescent lighting throughout the ship. Interior communication and signal circuits as well as various control circuits are fed from one or more separate switchboards which permits centralized control of most of the special power requirements.

Auxiliary sea-water cooling systems are not pertinent to your problem but of course are used extensively on submarines.

Fresh Water for drinking, washing, cooking, etc., is obtained from sea water by distillation. The distilled water is stored in tanks. Air is applied to pressurize the tank and distribute water through the piping system. As a rule of thumb, the capacity of the distilling plant provides the fresh water required by the propulsion plant, plus 20 gallons per man per day.

Sanitary disposal is accomplished by draining showers, wash basins, toilets, etc., to sanitary tanks. The tanks can be emptied by opening an overboard discharge valve and using air pressure to blow the contents overboard. The tanks are sized so that this evolution need be accomplished no more than once a day. I don't expect that your problem will be this simple.

Air Systems. The principal requirement for high-pressure air on submarines is to blow the water from the ballast tanks in order to attain positive buoyancy and permit the submarine to operate on the ocean's surface. Air for this purpose is stored in flasks at 3000 psi. To fill this storage system, the submarine is equipped with two or more electric-motor-driven air compressors. The capacity of these is a function of the volume of storage banks which in turn are sized to allow the submarine to blow ballast tanks several times without recharging the storage banks. Air from this system is used at reduced pressure for many other purposes. I've mentioned blowing sanitary tanks and pressurizing the fresh water system. It is also used to transfer lubricating oil and hydraulic oil from storage tanks to operating systems, in the torpedo firing system, for certain propulsion plant functions and some pneumatic control systems. Incidentally, during periods of long submergence the ambient pressure within the submarine builds up as a result of certain of these blowing operations which must be followed by venting the tanks into the submarine, as well as by air leakage. To maintain a comfortable ambient pressure, the high-pressure air compressors are occasionally run submerged to bring the ambient pressure down by returning air to the high pressure storage banks.

I'll discuss the ventilation system in conjunction with air purification in a few moments.

The hydraulic system on submarines is quite extensive and perhaps best illustrates the numerous problems faced by the designer. Hydraulic power is used to move the diving planes and rudder; to raise and lower periscopes, radar and radio masts; to motivate the many large valves in the ship's ballast system, the trim control system, and the propulsion system; to motivate components of the torpedo tubes and missile system; and to actuate many other items ranging from the anchor windlass to the ship's air whistle. The system is in general based on the recognition that certain components, such as the rudder, represent a continuing demand which must be met by the pump capacity, while other functions, such as raising the periscope, are step demands which can best be met by the stored energy of the hydraulic accumulator.

The designer must first consider the functions which require power actuation and tentatively determine whether these can best be met by hydraulic power or by some other means such as electrical, electro-mechanical or pneumatic motivation. Having reached some tentative determinations, he then considers these demands as "continuous"--which affect the pumping capacity of the system--or as "step" demands which affect the size of the accumulators. He now is prepared to picture the system in diagrammatic fashion, to roughly account for pump size, accumulator size and to approximate pipe size on the basis of estimated flow requirements and pressure drop.

The designer now must select system pressure. In this choice, he must bear in mind space and weight requirements, heat loss, noise, reliability of components, ease of maintenance, and cost.

Currently, we are using 3000 psi in most hydraulic systems, although in some ships a 1500-psi system has been found best suited.

With pressure established, the initial assumptions are refined and he is approaching firm sizing of principal components. He may now find that some one function adds so much to the pumping demand that an additional pump is required. Since there are many advantages to keeping the several hydraulic pumps identical, this one function may represent a costly increase, particularly from the space viewpoint, and so this function will be re-examined for possible operation in some other manner. Similarly, the designer must group the "step" demands in an intelligent fashion, keeping clearly in mind the normal and emergency operating conditions in order to arrive at accumulators of adequate size but not over-demanding in space and complexity. He may also find that the system pressure selected causes a difficult problem in designing certain of the actuators. This may occur when the maximum or minimum speed of movement of the actuated item is critical. It may be prudent to use another technique for actuating this item.

Finally, he must weigh these several decisions against the ship as a whole and the overall operational requirements. For example, the lack of efficiency in hydraulic system design will add to the electrical load and the air-conditioning load. One noisy component may increase the range at which this submarine can be detected or may reduce the effectiveness of the submarine's own sonar system by raising background noise. The final selection is always a compromise--hopefully always the best compromise.

Now, gentlemen, a word about atmospheric habitability. By centuries of evolution, man has established himself as a creature of the earth's surface at or near sea level. This is where he functions best. When he departs from this environment he soon finds himself in difficulty. He can continue or proceed only with the aid of external protective devices such as oxygen supply on high-altitude flight or helium-oxygen for deep-sea diving. The standards for submarines have been established after review and evaluation of all available evidence and represent a consensus of the best medical minds in the country in relation to the particular problem. These constitute an informed estimate of that level of contamination which will not adversely impair health nor cause an unacceptable reduction in performance. All such standards are interim standards and represent a compromise between what is clearly feasible from an engineering viewpoint and what appears desirable from the standpoint of health and performance. The ultimate goal is a respirable atmosphere as free of contamination as ambient air.

Ambient air at sea has an oxygen content of 20.9 per cent. The current level for oxygen in submarines is 20 to 17 per cent, but the goal should be to maintain 20 per cent oxygen as long as possible. Except in extreme emergency the oxygen should not be allowed to fall below 17 per cent.

Ambient air at sea has 0.4 per cent carbon dioxide. Experiments have shown that an elevation to 1 per cent is acceptable, but the goal should be to maintain less than 1 per cent as long as possible.

Ambient air at sea has no carbon monoxide. The current submarine standard is to maintain less than 25 parts per million.

Until the advent of nuclear power the ability of a submarine to remain completely submerged was limited by the energy storage capacity of her lead-acid storage batteries. Except in rare instances, the submarine had completely expended battery energy before lack of oxygen or excess of carbon dioxide became a critical problem. It was then necessary for the submarine to surface or use the snorkel to obtain air necessary for combustion in diesel-driven generators. Hence, atmospheric habitability was not a serious problem.

It wasn't until nuclear power came along, however, that the real problems of maintaining a satisfactory atmosphere in a submerged submarine for periods on the order of 30 days were attacked. At the present time, the only consideration limiting the period of submergence of a nuclear-powered submarine is our ability to maintain satisfactory atmospheric habitability. You have all read in the newspapers that USS Skate and USS Nautilus both remained submerged for periods in excess of 30 days and that USS Seawolf was submerged for 60 days.

The principal factor limiting submerged endurance in these ships was the ability to replenish oxygen. The earlier nuclear-powered submarines carried a large quantity of gaseous oxygen stored in flasks at high pressure. Such a system is costly from the weight and space viewpoint. The Navy has developed an oxygen generator using the principle of electrolysis, which will enable us to generate oxygen from distilled water. Such units are being installed in submarines currently under construction. This means that oxygen will no longer be the controlling item of atmospheric habitability and the ceiling of submerged endurance will be raised another notch.

Figure 7 shows a diagrammatic representation of the ventilation and air purification system we are now providing in nuclear-powered submarines. In general terms, it consists of a means of drawing air from the extremities of the ship and passing it through certain purification equipment which I will describe later. Circulation of a small portion of the total volume of air in the submarine on a continuing basis is sufficient to maintain the desired degree of purity. In addition to this overall circulation system, the temperature and humidity control in the several areas of the submarine are maintained by individual recirculation systems. The oxygen generator is designed to be operated continuously while the submarine is submerged, and the output of this unit has been sized to replenish the oxygen consumed by the personnel on a continuing basis. The one undesirable product of this process, namely, hydrogen gas, is forced into the solution with sea water and carried overboard. During the periods of start-up and shut-down, the entire system must be carefully blanketed with nitrogen to minimize explosive hazard. In the application you are considering here, since distilled water is not readily available in quantity, a more satisfactory solution will probably be to provide gaseous oxygen stored at high pressure.

The problem of maintaining an acceptable concentration of carbon dioxide has been solved by a CO_2 scrubber. The CO_2 scrubber is a regenerative
05 closed system whose operation is based on the fact that a cool amnine solution readily absorbs carbon dioxide while a hot amnine solution gives up part of its carbon dioxide content. The amnine solution used in our scrubbers contains Monoethanolamine, a chemical compound derived from an ammonia base. In the purification cycle, air containing CO_2 is drawn from the ship's atmosphere and passed through an absorber tower where it is sprayed with cool amnine. The air is then washed with fresh water and sodium-bisulphate, to remove amnine vapor, and returned to the ship's ventilation system. The amnine, which has now absorbed CO_2, is heated in the stripper tower. Amnine and water vapor and CO_2 are given off and these are then passed through a cooler where the vapors are condensed. The remaining CO_2 gas is compressed and discharged overboard. As in the case of the oxygen generator, this unit is run continuously while the submarine is submerged and is sized to maintain the submarine atmosphere at less than 1 per cent CO_2 on a continuing basis. As standby equipment for the CO_2 scrubbers, we do carry an emergency supply of pelletized lithium hydroxide. This chemical is contained in sealed canisters each about the size of a large loaf of bread. When required

for use, the ends of the canisters are opened and several canisters placed in a hopper. A small ventilation fan would then take air from the ship's atmosphere and pass it through the open canisters. Experience has shown that approximately 1/10 lb of lithium hydroxide per man per hour would be required to maintain a satisfactory level of CO_2. In your application, it may well be more expedient to provide a large supply of a chemical such as lithium hydroxide, rather than to utilize a regenerative-type CO_2 scrubber.

Three other equipments are used to maintain pure air in our nuclear-powered submarines. One is known as a hydrogen and carbon monoxide burner. The CO_2 removal technique will not handle the carbon monoxide problem, and submarines have long had a problem with hydrogen gas which is given off by the lead-acid storage batteries during the charging cycle. The hydrogen and carbon monoxide burner is simply a unit which heats the ship's air and passes the heated air through a catalyst bed where oxidization occurs. The outcoming air is then cooled to condense water vapor and passed over a lithium hydroxide bed to absorb carbon dioxide before returning to the ship's atmosphere.

Another item is the electrostatic precipitator which is used to remove grease, smoke and other fine particles which are suspended in the air. The precipitator consists of plates charged to high voltage with direct current to attract dirt and grease particles. The plates are removed at intervals and washed clean in the galley sink.

Another item of the system is a charcoal filter bed. This is principally for the purpose of removing odors from the air. In some installations we have provided a single large bed of charcoal, while in others we have spread it around through the ventilation system in convenient locations. Charcoal filters are also installed in the sanitary tank vents and in the ventilation recirculation system in lavatories.

In order to insure that the atmosphere in all compartments of the submarine is being maintained in a satisfactory condition, we have a centrally-located atmosphere analyzer. This unit, through small tubing, can take a suction from any compartment in the ship and analyze the content of this air for oxygen, carbon dioxide, carbon monoxide, hydrocarbons and the presence of freon or phosogene gas.

In addition to maintaining the proper purity of air, submarines are also designed to maintain an effective temperature of 78° F or less in all living and control spaces. This condition is further defined as 85° dry-bulb and 50 per cent relative humidity. This is the design condition for living in control spaces, but not for machinery spaces where we have a large quantity of steam machinery. For the purposes of heating and cooling, the nuclear submarine is divided into zones which are serviced by separate loops since they represent a particular type of heating load. The recirculating system for each of these zones consists of a fan taking suction across a cooling coil and discharging through an electric heater before returning the air to the ship. The cooling coil is controlled by a humidistat and a thermostat while the heating coil is, of course, controlled only by the temperature in the zone. It is evident that when the ship is surrounded by cold water, the cooling required to maintain a proper relative humidity must, on occasion, be supplemented by heat in order to maintain proper temperature. The number of recirculation zones is a function of the particular arrangement within the ship and the nature of the equipment located in the several areas.

For many years the Navy has favored chilled-water cooling coils for shipboard ventilation rather than the direct expansion of freon. The freon refrigeration piping is therefore limited and localized. This proved very important in atmospheric habitability because freon, when in contact with hot surfaces, gives off phosogene gas. We are taking pains to make the freon system absolutely tight.

Many other gases and chemical products were at least a matter of concern to the medical people. Experience has proven that these either are not generated in significant quantities or are removed to an acceptable extent by one of the systems discussed above.

A lot has been said and written about the interior decoration features of nuclear submarines. I'm sure I don't need to emphasize the importance of this aspect of habitability. We have done a great deal to make the living spaces aboard submarines more attractive, useful and practical.

Before closing, I would like to mention briefly a technique that has become quite prominent in submarine design and construction. When one is trying to achieve the best arrangement of a lot of equipment in a small space, we have found it necessary and profitable to resort to mock-ups. For design purposes, we are using 1/4 scale--quarter of full size--mock-ups to achieve optimum utilization of space. This mock-up is located convenient to the designers and is fabricated in sections which can be moved apart for better accessibility. The degree of detail represented is a function of the critical nature of the arrangement in a particular area. All major runs of piping and wiring are represented, however. Plastics are used quite extensively.

Completely separate from this 1/4-scale mock-up, which is principally a design tool, we utilize a full scale mock-up, constructed principally of wood, to aid the production personnel in the assembly of complicated or confined areas. For example, piping bends can be templated in an accurate mock-up and the fabricated pipe checked, thereby relieving the congestion in crowded spaces during ship construction.

FIGURES

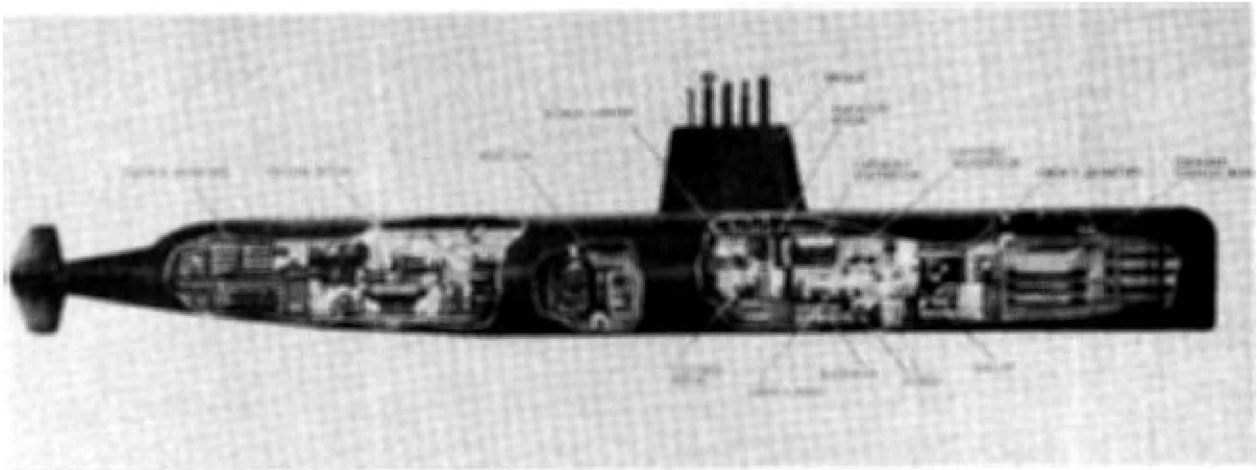


Fig. 1

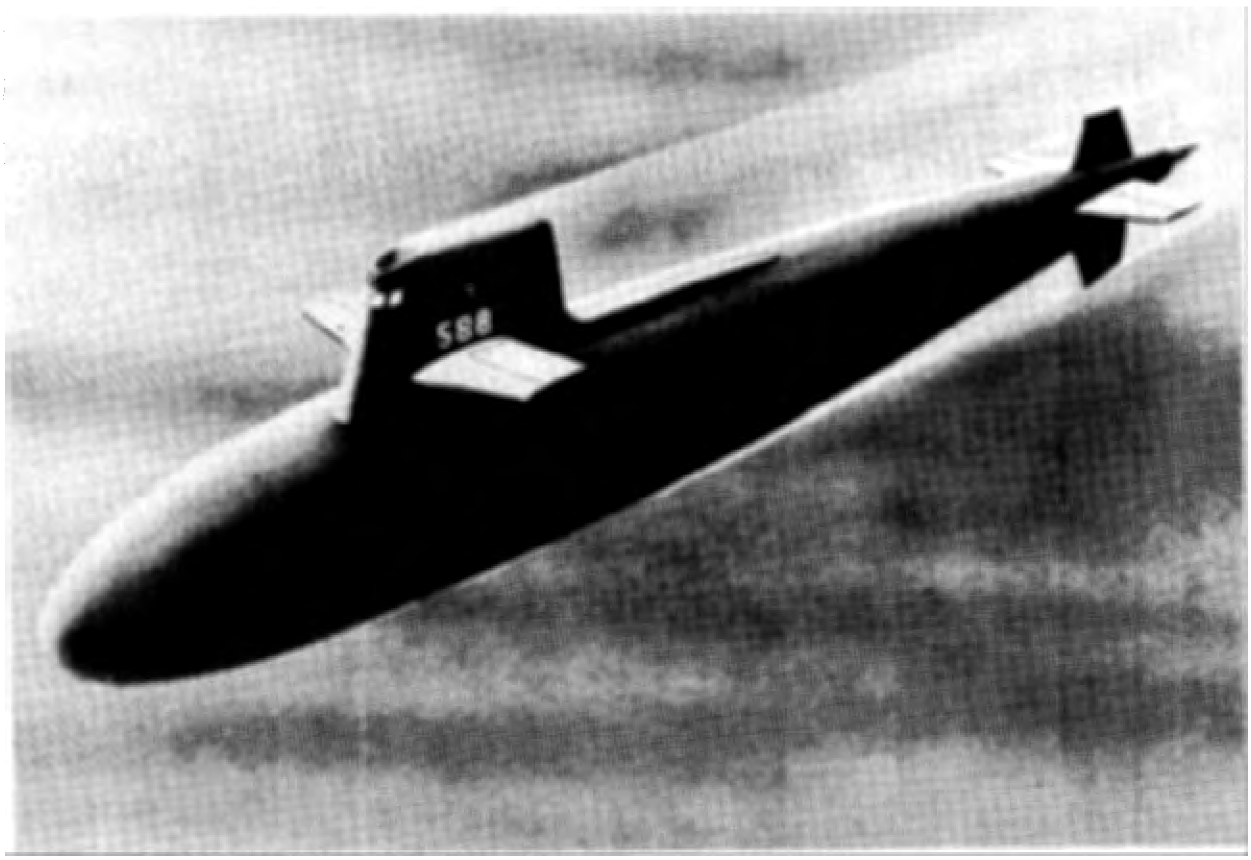


Fig. 2

R-341
3-26-59
540

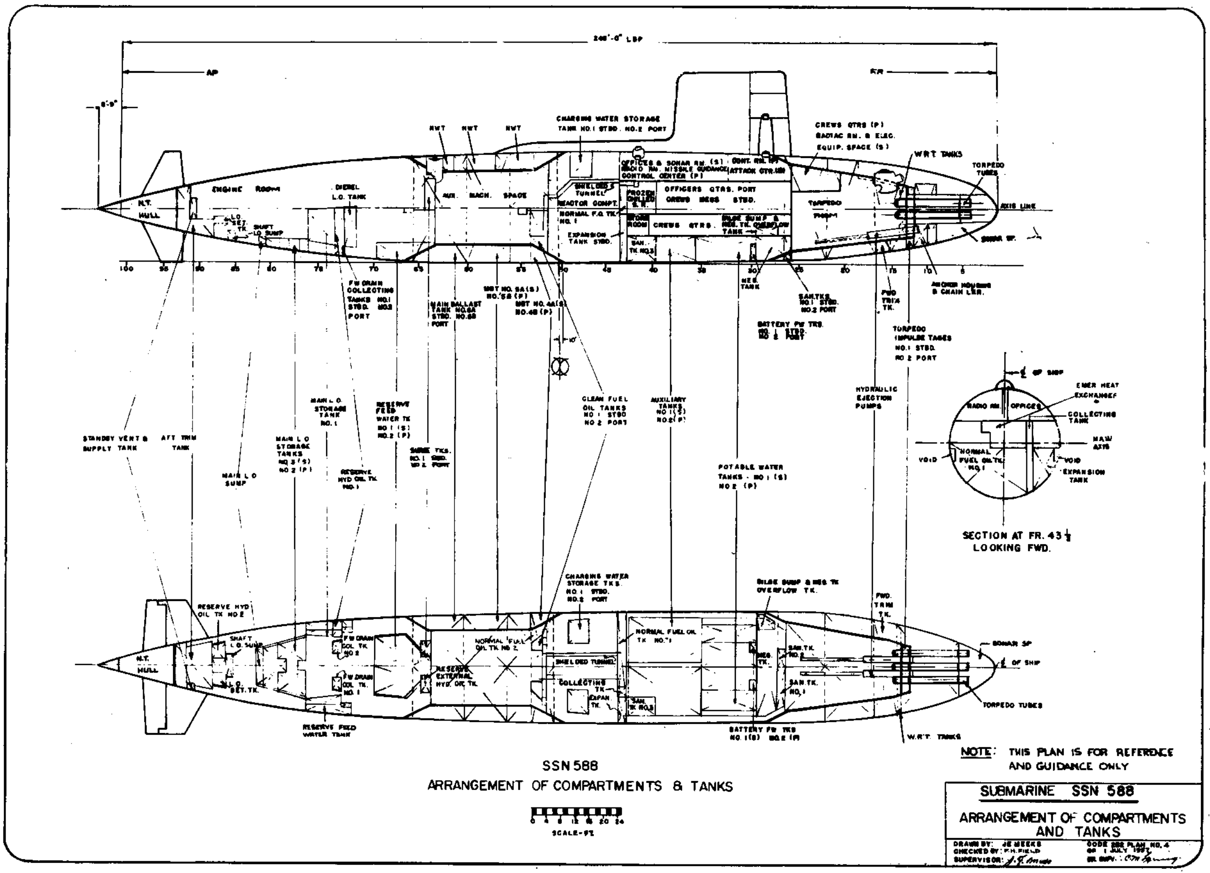


Fig. 3

Fig. 4

FACTORS IN UTILITY DESIGN

Reliability

Noise

Maintenance requirements

Cost

- Initial
- Ultimate

Space

Emergency provisions

Weight

Heat loss

Fig. 5

SUBMARINE UTILITY SYSTEMS

Electrical

- Auxiliary power
- Lighting
- Interior communication and control

Auxiliary sea water cooling

Fresh water

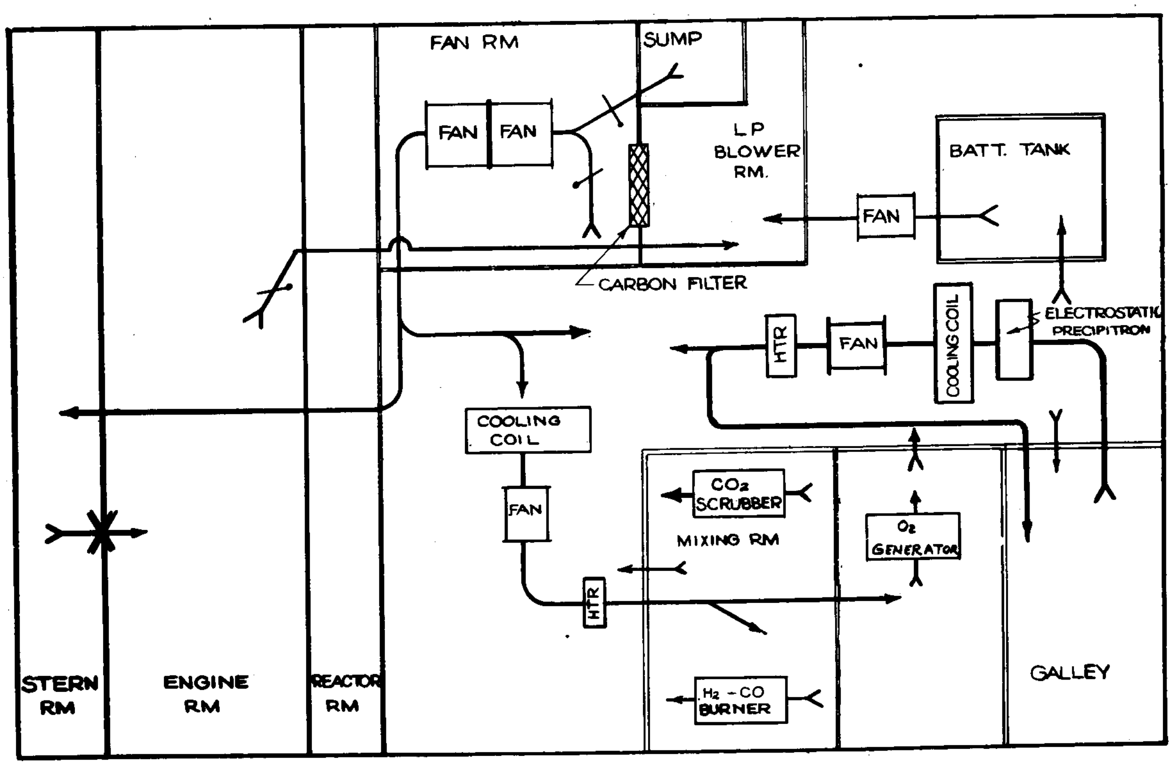
Sanitary disposal

Air systems

Ventilation

Hydraulic

Fig. 6



Typical air distribution

Fig. 7

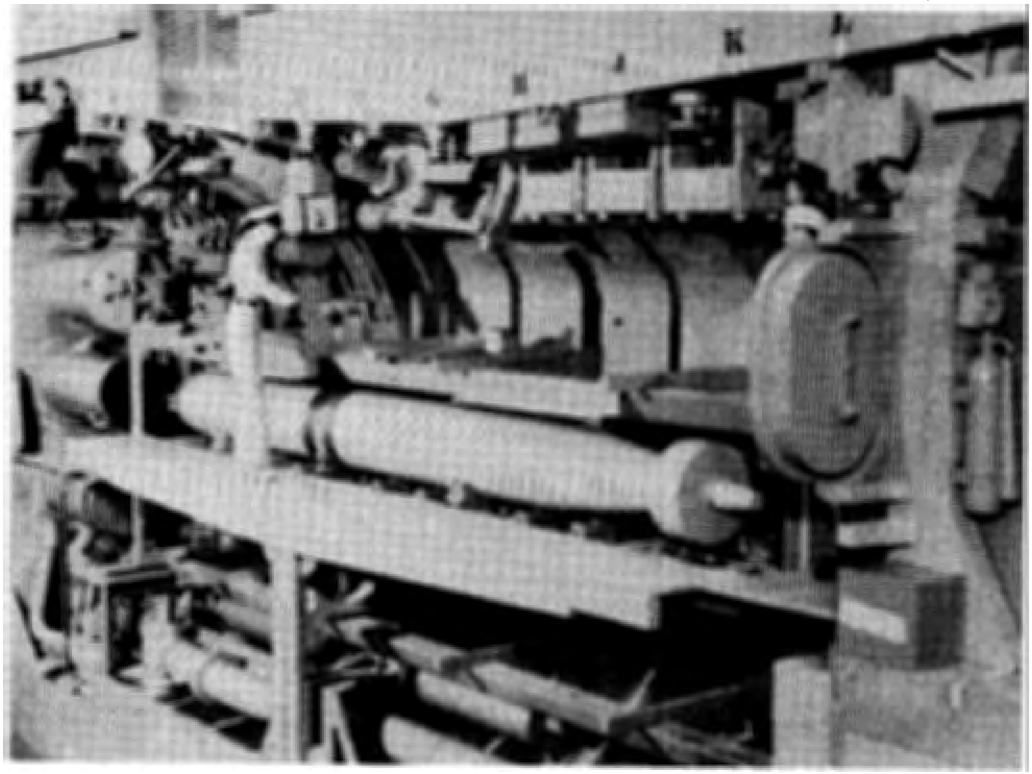


Fig. 8

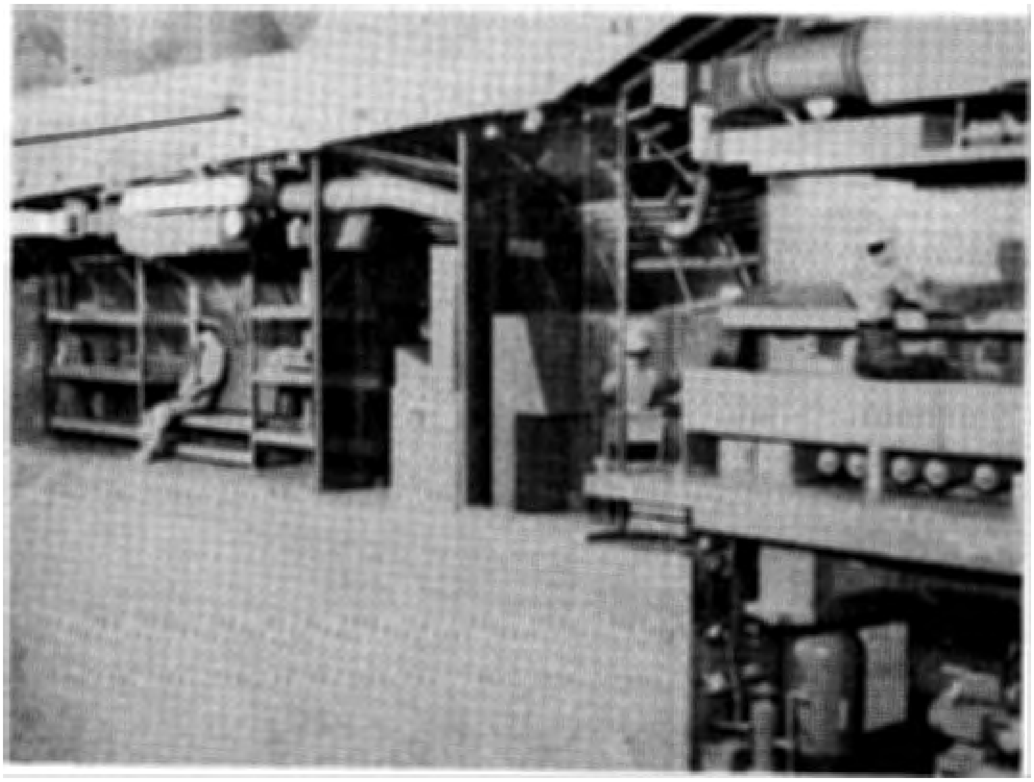


Fig. 9

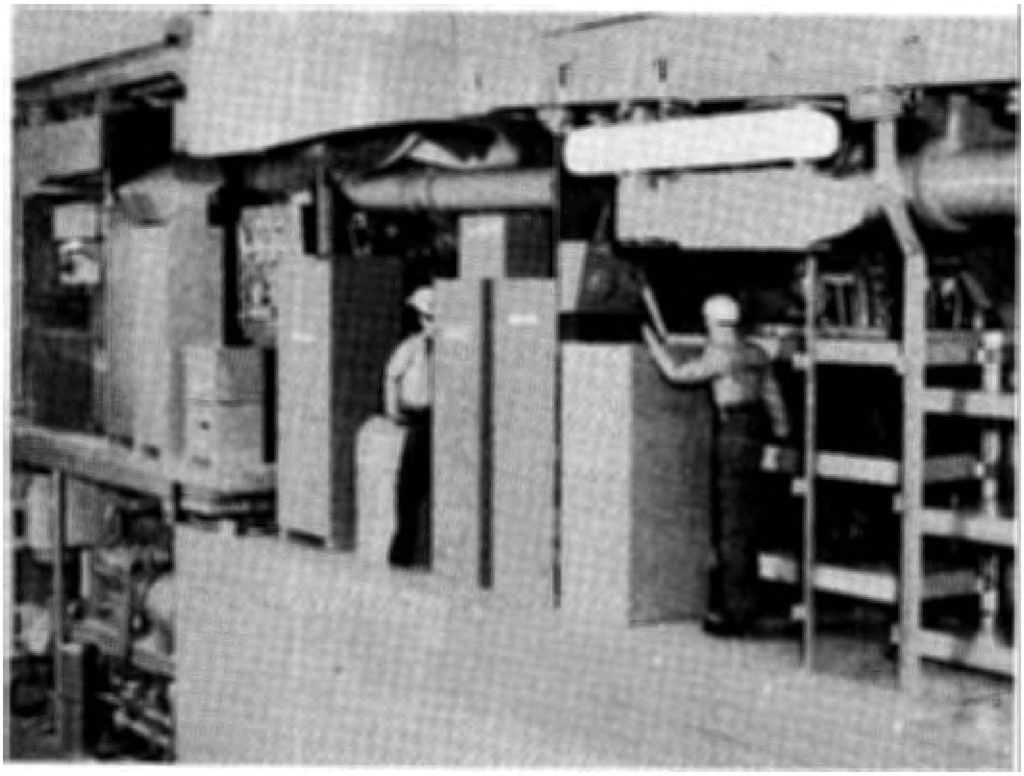


Fig. 10

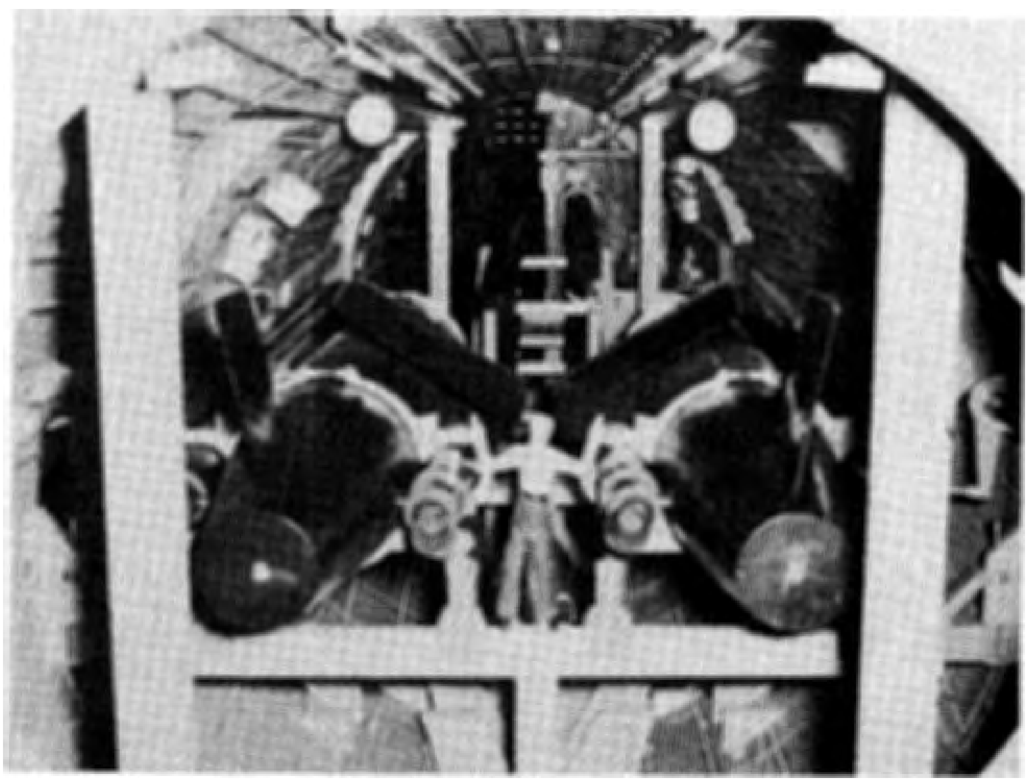


Fig. 11

COMMENTS ON ELECTRICAL POWER SUPPLIES FOR UNDERGROUND SHELTERS

John Huth
The RAND Corporation

Let me preface my remarks by saying that my background lies primarily in the field of power supplies for space vehicles. Consequently, in accepting an invitation to speak at this conference I tried to impress upon its organizers the fact that I am almost completely unfamiliar with their research. I must therefore begin by discussing those properties that I would consider highly desirable for a civilian shelter power supply.

1. It should be capable of standing idle for long periods without becoming incapacitated or hard to start.
2. It should be capable of operation by untrained personnel.
3. It should be relatively insensitive to shock.
4. It should not produce obnoxious fumes.
5. It should not present a heat removal problem.
6. The output must be in a suitable form; i.e., preferably 110-V AC at 60 cycles.

Aside from some very special circumstances such as an underground shelter near a natural source of steam, we will be limited to chemical or nuclear systems. Starting with chemical systems, the old stand-by's are diesel engines and turbo-generators. Neither can merely be left standing idle and unprotected for long periods and then be expected to start at the push of a button. However, they might for example be protected by a nitrogen atmosphere. Otherwise, they must be run fairly often. Anyone interested in this subject might profitably look into Navy methods of mothballing ships. I would feel quite certain that either turbo-generators or diesel-driven units could be designed and mounted to withstand fairly severe earth shocks. I would be less certain about the ability of non-mechanics to start and operate either and, as a dweller in the Los Angeles basin, I am reminded almost daily of the noxious and dangerous fumes that can arise from the use of hydrocarbon fuels.

An alternate scheme would be to use batteries. Very long shelf lives could be anticipated by utilizing primary cells in which the electrolyte is not introduced until just prior to usage.(1) One of the better conventional electro-chemical systems is the silver-zinc cell, capable of delivering about 50 watt-hours per pound. There are no fumes to be vented, amateur operation is quite feasible, and again shock should be no problem. Also, in all common electrochemical systems the free-energy and heat of the reaction are very close.(2) This merely means that little heat is normally generated in the discharge of a battery (almost all will arise from irreversible ohmic losses). However, a primary cell can be tested only at the expense of some of its ultimate capacity (and destruction of the automatic activator), and low-voltage DC may not be directly applicable. If the latter is true, static transistorized inverters are now available and exhibit efficiencies of the order of 80 per cent.(3)

Another perhaps less familiar type of battery is the so-called hydrogen-oxygen fuel cell.(4) This is a battery-like device in which the electrodes behave electrochemically as if they were made of hydrogen and oxygen. Such a concept is not at all new, having been discussed by Ostwald at the founding of the Electro-Chemical Society. However, reduction to a practical device is a fairly recent achievement. Hydrogen and oxygen are introduced to their respective electrodes (made of some inert material such as carbon or platinum), where they are ultimately adsorbed on the surfaces. The overall cell reaction, $2H_2 + O_2 \rightarrow 2H_2O$, proceeds in the presence of an alkaline electrolyte. Essentially the cell produces an inverse electrolysis with about 60% of the heat of the reaction appearing directly as electricity. The electrodes and electrolytes remain unconsumed, the energy coming from the consumption of feed gases stored in separate tanks (up to about 300-watt-hours per pound of feed gases plus containers is currently attainable). There are no fumes, the only product of the reaction being water. As compared with ordinary primary cells, fuel cells offer considerably greater energy storage capacity. However, there is a definite heat rejection problem, and the wide flammability limits of hydrogen may make storage risky in an area subject to shocks. In theory, other gases can also be combined electrochemically, but only the reaction using quite pure H_2 and O_2 has as yet achieved any practical success.

Let us now turn to nuclear energy sources. Here we have a choice between isotopes and reactors. I am going to dismiss the former by merely pointing out that there will probably never be enough radio-active waste material to warrant considering its use on any really large scale. Regarding the latter, a reactor has the obvious advantage of a very long lifetime. A reactor can be quite compact and safe, and shielding should not pose too great a problem. However, a reactor is basically a source of heat and if we use any of the conventional conversion schemes (turbines, etc.), we again have need for special storage, etc. However, there are direct-conversion schemes under development which might eliminate altogether the need for moving parts. I refer to the possibility of using a homogenous reactor with either thermocouples or thermionic converters. One of the prime problems with such a system would be that of heat rejection.

In conclusion, I would say that development of an altogether suitable stand-by electrical power source for underground shelters must first await specification of "ground rules" on types of operators, frequency of testing, etc. Secondly, one must consider in some detail problems of heat rejection and venting of waste products.

REFERENCES

1. Proceedings of the Tenth Annual Research and Development Conference, Power Sources Division, Signal Corps Engineering Laboratories, Fort Monmouth, New Jersey, May 23, 1956.

2. Vinal, G. W., Storage Batteries, Wiley and Sons, 1949, pp. 185-186.

3. Moore, D. W., and R. D. Gates "Special Report on the Solid Inverter," Western Aviation, November 1958, pp. 6-7.

4. Potter, E. C., Electrochemistry, Principles and Applications, MacMillan, 1956, pp. 402-408.

DISCUSSION

MR. PEAVY: Have you compiled anything for the amount of space needed for this type of battery that you are talking about? Space is quite a thing underground.

DR. HUTH: The third slide showed volume figures for some of the typical battery types. For the fuel cell itself, the volume figure would be quite comparable to a diesel generator. The storage problem of the gases is a little bit sticky. Of course, depending on the pressure used, the volume would vary; but it would certainly be larger than for ordinary diesel fuel, for example.

MR. WILLIAM W. PLEASANTS (RCA, Moorestown, New Jersey): Can you tell us what is the largest installation now operating on this electrochemical cell?

DR. HUTH: Fuel Cell?

MR. PLEASANTS: I believe the last thing you recommended was an electrochemical cell.

DR. HUTH: What I said at the end was that I thought the electrochemical system would look good for this operation. In particular, I mentioned the silver-zinc cells.

MR. PLEASANTS: What I'm wondering is are there any sizeable installations now operating?

DR. HUTH: I think so, but not for this application. The largest application of the batteries that I would know of offhand would be in submarine batteries, which are quite large. I would presume these things would certainly be delivering kilowatts for several hours.

PRIMARY ENERGY SOURCES

TYPE	ENERGY RELEASE ERGS/gm	ENERGY RELEASE CALORIES/gm	ϵ, FRACTION OF ANNIHILATION ENERGY
CHEMICAL (HYD-OXY)	1.33×10^{11}	3.19×10^{3}	1.48×10^{-10}
SUPER-CHEMICAL ($H + H \rightarrow H_2$)	2.2×10^{12}	5.25×10^{4}	2.45×10^{-9}
NUCLEAR FISSION	7.1×10^{17}	1.69×10^{10}	7.9×10^{-4}
THERMONUCLEAR	3.6×10^{18}	8.6×10^{10}	4×10^{-3}
MATTER ANNIHILATION	9×10^{20}	2.14×10^{13}	1.0
SOLAR	3.3×10^{2} $\frac{\text{CALORIES}}{\text{SQ METER}}$ $\frac{1}{\text{SEC}}$		

Fig. 1

ELECTRICAL POWER SOURCES

CHEMICAL

- DIESEL-GENERATOR
- TURBO-GENERATOR
- BATTERIES
- FUEL CELLS

NUCLEAR

- ISOTOPE
- REACTOR

Fig. 2

ELECTROCHEMICAL SYSTEMS

TYPE OF CELL	LIMITING THEORETICAL PERFORMANCE (WATT-HR/LB)	CURRENTLY AVAILABLE PERFORMANCE (WATT-HR/LB)	(WATT-HR/CU IN.)
LEAD-ACID	75	15	1
NICKEL-CADMIUM	92	15	1
ZINC-SILVER	176	60	3
HYDROGEN-OXYGEN	1700	300	4.5

Fig. 3

PERFORMANCE CHARACTERISTICS OF A SEALED NICKEL-CADMIUM CELL

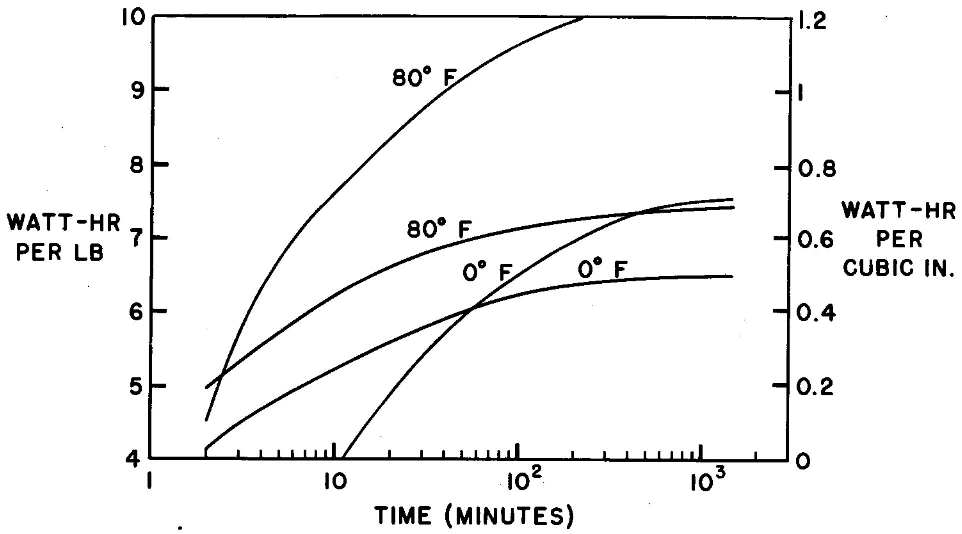


Fig. 4

FUEL – CELL

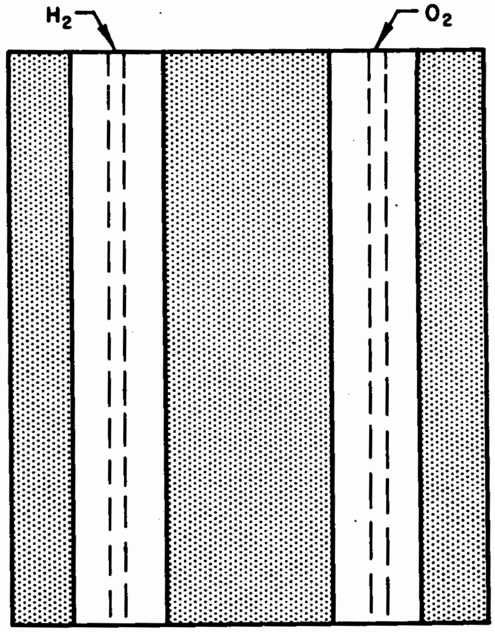


Fig. 5

SPECIFIC POWER OF PURE ISOTOPES

ISOTOPE	SPECIFIC POWER OF PURE ISOTOPE (WATTS/LB)	SPECIFIC POWER OF ATTAINABLE ISOTOPE COMPOUND (WATTS/LB)	HALF-LIFE	SOURCE
STRONTIUM 90	421	42.5	28 YEARS	FISSION PRODUCT
PROMETHEUM 147	157	24.2	2.6 YEARS	FISSION PRODUCT
POLONIUM 210		6.4×10^4	138 DAYS	NEUTRON IRRADIATION OF BISMUTH 209

ESTIMATED FISSION PRODUCT POWER FROM NUCLEAR POWER INDUSTRY

YEAR	CUMULATIVE INSTALLED REACTOR POWER (MEGAWATTS)	APPROX TOTAL BETA AND GAMMA BY-PRODUCT POWER (KILOWATTS)
1965	3,600	2,000
1975	84,000	50,000

Fig. 6

UTILIZATION OF ISOTOPE POWER (A)

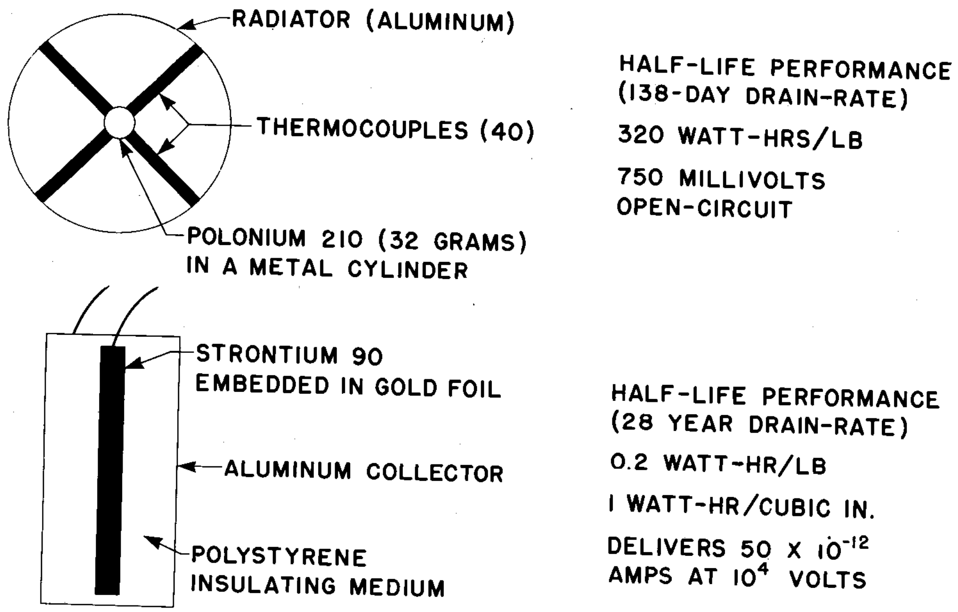


Fig. 7

DIRECT CONVERSION REACTOR

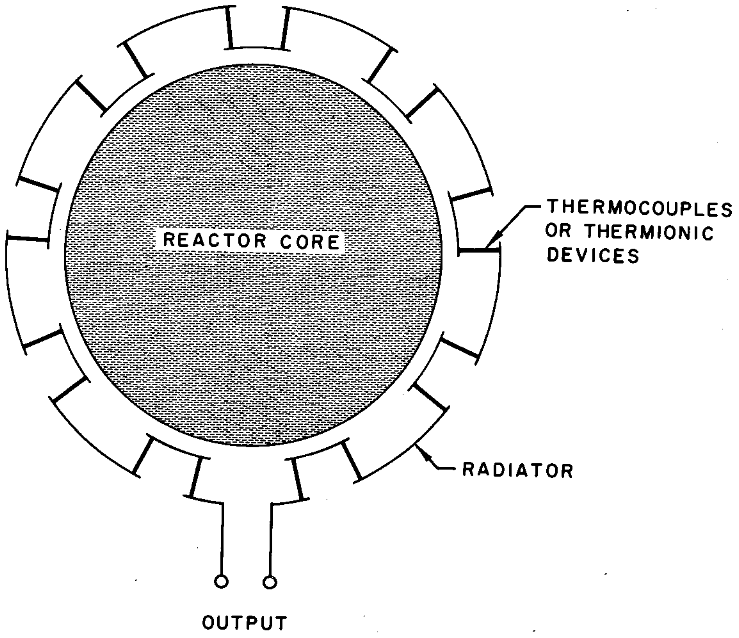


Fig. 8

COLLECTIVE PROTECTION AGAINST CHEMICAL, BIOLOGICAL AND RADIOLOGICAL WARFARE AGENTS

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Protective Construction Branch
Office, Chief of Engineers
U. S. Army

The requirement for ventilation air in underground structures and the protective aspects of obtaining this air is the subject of this paper. However, to discuss this subject we must consider the over-all problem of obtaining maximum protection of structure interiors and its occupants against the infiltration or migration of contamination. It is also advisable to consider all types of contamination that could affect the occupants and the mission of a structure and not just the obvious and current hazards of radioactive fallout. From the viewpoint of the mechanical engineer, the hazards of fallout are the simplest to combat, as it is readily detectable with proper instruments and easily removed from the ventilation air. The more modern nerve and sensory chemical contaminants are more difficult to detect and remove from the air, and the biological contaminants are not detectable by instruments. It is only the strict wraps of security and the newness of the nuclear weapon that has placed fallout in the forefront, and chemical and biological agents in the background. It would be taking the "ostrich" approach to the problem to neglect the chemical and biological effects.

The problem of providing protection against chemical, biological and radiological airborne contaminants involves not only air filtration but also personnel decontamination facilities, internal pressurization, air conservation, instrumentation and, most important of all, related operational procedures. Let us go over this one point, which I feel is most important, namely, operational procedures. It is one thing for the designer to know why he installs certain equipment but it is another thing to assume that the occupants and operating personnel will understand the purpose of such equipment. It has been found, in many instances where protective structures have been visited, that responsible operating personnel had no idea what these "big boxes" (the filters) are for or why such narrow corridors with showers were placed at the entry of the structure. This fact is brought out only to stress that the responsibility for educating the occupants of a protective structure lies with the designer. Currently this is being done by preparing an operational procedure at the time that designs are prepared.

Let us now turn to the physical requirements of a protected structure.

First, it is essential that a continuous supply of filtered air be introduced into the structure. This is essential to the prolonged occupancy of the structure and is required for developing an interior pressure of approximately 1/2 inch of water to reduce the possible infiltration of airborne contaminants and to provide a means of controlling the outward flow of air through decontamination chambers. The filtration of air is accomplished with special filter units developed and produced by the Army Chemical Corps. These filters are comprised of two parts, one of which filters out particulate radioactive fallout materials and biological warfare agents. The other retains and neutralizes chemical warfare agents. These will normally be procured as a single unit assembled in a plywood housing. The capacities and weights of these combined units are as follows: M-14, 600 cfm, 310 lbs; M-15, 1200 cfm, 570 lbs; M-16, 2500 cfm, 1090 lbs; and M-17, 5000 cfm, 2240 lbs. The size of these units ranges from 2 ft square by 3 ft long to 4 ft square by 5 ft long.

The resistance of these filters at their rated capacities is three inches of water, but five inches of water should be used for design purposes. Due to the bulk and excessive weight of these filters, ample space and special handling equipment is required for their installation and ultimate replacement. In an effort to localize the potential hazards resulting from the entrapment of radioactive particles in the filter, it is advisable that these units be placed in an area away from personnel activity. Also, these filters should be located near an exit in order to reduce the hazard of contaminating the occupied areas of the structure when the filters are removed from the system for replacement. It might be well to mention in passing that these filter units cannot be installed in a vertical position, as might be desired, in an effort to conserve floor space. This positioning limitation is due to the charcoal bed configuration within the gas filter unit which may permit the charcoal to settle, allowing air to bypass the charcoal at that point. However, in an effort to conserve space and also reduce maintenance, it may be permissible to separate the particulate filter from the charcoal filter. I use the expression "it may be permissible" solely because we have never had an occasion to do this and have not discussed it with the Chemical Corps, the agency which has primary responsibility for the filters. This separation of filter components would permit the replacement of the particulate unit without need of replacing the more expensive and bulky gas unit. Consequently, it is worthy of consideration. A simplified ventilation system containing the CBR filters is illustrated by Fig. 1. It will be noted that this system has a means of obtaining ventilation air without passing through the filters. This bypassing was at one time considered advisable in order to conserve filters. However, modern concepts of warfare are reducing warning time to such an extent that no time is available for changing from unfiltered to filtered air. This factor, together with the undetectable characteristics of biological contaminants, makes it advisable to filter ventilation air at all times.

Figure 2 illustrates the personnel decontamination facility. This is a dual facility primarily designed for male and female personnel. The physical size of the decontamination facility and the requirement for duplication will be dependent upon the type of structure, the mission to be accomplished, and the number of persons that may utilize the facilities during any one period. This decontamination facility is basically a development of the Chemical Corps and is a result of numerous tests of personnel contaminated with a biological simulant. The cross-sectional area of the undressing and showering area is 3 ft by 7 ft, which has been determined as adequate space for undressing, while at the same time reducing the quantity of air needed for scavenging this area. This quantity of air is approximately 400 cfm and is obtained by maintaining a differential pressure of 1/10 inch of water across the membrane doors. In the absence of detection equipment for BW contaminants it was necessary to determine a time in which washing with soap would safely decontaminate an individual. This time was established as three minutes, assuming a shower having a waterflow of five gallons per minute. If individuals can be monitered, as with chemical and radiological contaminants, undressing and showering may or may not be necessary, yet the facilities are available. This, together with an allowance of three minutes each for undressing and redressing, would permit an entry time of nine minutes per person.

The membrane door illustrated in Fig. 3, also a development of the Chemical Corps, provides a means of entry through a minimum opening, and also permits an outward flow of air over the entire cross-sectional area of the decontamination chamber for scavenging. This door is generally as shown here and is made of a rubber-nylon mesh, better known as girdle material. These points are covered in detail in Engineering Manual 1110-345-461.

Internal pressurization is also a requirement in protected structures, partly to offset infiltration. While there is a lesser infiltration problem in underground installations, pressurization is still essential in such facilities for the free movement of air through various areas of the structure. Pressurization is obtained by restricting the outward flow of exhaust air. Consequently, it is necessary to provide air seals on all conduits, roadway drains, service trenches, etc., where air may escape. The free air flow through the decontamination area is accomplished by obtaining a pressure differential of approximately 3/10 inch of water above atmospheric directly inside of the exterior door, which in many instances is a blast resistant door. This first stage of pressurization is accomplished with an adjustable anti-back-draft valve. Either 400 or 800 cfm will be the quantity of air involved, depending upon whether a single or double decontamination facility is provided. The next stage of pressurization occurs at the membrane doors where 400 cfm of air is passed with a pressure differential of approximately 1/10 inch of water. This pressure staging will provide a 4/10 inch of water pressure throughout the remainder of the structure. The main thing to consider in obtaining pressurization is the control of all potential areas of leakage. This will eliminate the need for excessive quantities of filtered air over that required for proper habitability of an underground installation. The optimum quantity of air introduced into a protective structure should not exceed 10 cfm per occupant. This will adequately provide for the physiological requirements of the occupants. It is not considered advisable to provide for the revitalization of air for periods when ventilation air may not be obtainable. Under circumstances when ventilation air must be shut off for short periods of time as a result of blast, the occupants can survive on the air available within an underground structure if maximum use is made of the air-conditioning and recirculation equipment. This, of course, is a function of space allocation. Under circumstances when ventilation air is shut off indefinitely due to blast damage, it would also be reasonable to assume that the generation of electric power stops due to the lack of combusion air. Consequently, operations would cease and personnel would vacate the facility as soon as possible and air revitalization equipment would be useless.

Combustion air for internal combustion engine generators and conventional boilers is a critical commodity in protective structures. This is not from the standpoint of filtration requirements but from the standpoint of maintaining continued operation of the equipment while being subjected to blast. The Corps of Engineers, through the cooperative effort of the Signal Corps, uses a radiation detection device illustrated by Fig. 4 for closing air intakes and exhausts in advance of the arrival of blast. While this system simplifies the problem of protection, there still remains the problem of maintaining operation of engine generators and instantaneously shutting off oil-fired boilers without causing damaging effects when a blast closure is figuratively "slammed shut" in the face of this equipment. The radiation detection device prepares the structure for blast by closing various openings and shutting off fuel lines, etc. This device or system is basically a pair of instruments that are sensitive to thermal and initial gamma radiation of a distant nuclear explosion and that will react to electrically trigger shut the protective closures as well as to provide a visual or audible warning alarm. The thermal and gamma detection devices together with control panel and other appurtenances are furnished by the Signal Corps as "Radiation Detection and Alarm System AN/FJW- 1 (V)." The gamma detector device uses an ionization chamber and electrometer circuit to detect the initial gamma radiation emitted at the time of weapon detonation. The radiation activates a relay that in turn opens secondary alarm and protective circuits, carrying current of 2 milliamperes. The thermal detector or triggering device consists of a phototube circuit sensitive to infrared. This device activates relay circuits on the rise time and duration of the thermal radiation pulse. A 1-millicurie cobalt 60 gamma source, pulsed by a solenoid assembly, is used to test the initial gamma detector, and an electronic photoflash circuit checks the operation of the thermal detector. Each detector is housed separately in a 5-in. diameter aluminum cylinder approximately 14 in. high and provided with mounting plates. The detector will be installed in such manner that maximum unobstructed view can be obtained over a horizontal azimuth range of 360 deg. The entire system is controlled and visually displayed on a master control panel that shows the activation and operation of the protective circuits and remotely controls the gamma and thermal testing devices. The system operates on standard 110-volt, 60-cycle alternating current at approximately 10 amperes. This system was developed as a sensing device for the operation of blast-closure valves. However, its use has much broader application where it is desirable to automatically shut off gas lines, fuel-oil lines, and other such services that may enter a protective structure. Also its use as a warning device, however short the time may be, will alert personnel of the arrival of blast and probable fallout.

The closure device illustrated in Fig. 5 and presently used in protective structures was initially designed for installation in an existing blast wall having air-intake openings 16-in. diameter in four horizontal rows of six each. Consequently, a seemingly complex mechanism has been developed and is continuing to be used in the absence of further development. This closure valve will handle approximately 5000 cfm of air with a flow resistance of 1.0 inches of water without consideration of the wall thickness. The closure valve as illustrated will close automatically upon firing of the explosive cartridge and is opened manually by a chain-operated reduction-gear assembly. The firing of the explosive cartridge occurs when the alternating current is shut off, either by the sensing elements or by normal power failure. The 110-volt alternating current is converted to low-voltage direct current by a transformer and selenium rectifier that continually energizes a capacitor or condenser. Upon failure of alternating current, a relay switch closes the circuit and the cartridge is fired by the potential in the capacitor.

With all this there still remains the unanswered question of how to obtain air for combustion and exhaust gases without regard to blast. Steam or hot water boilers cannot withstand any blast pressures and still be expected to function. The least that may happen is the collapse of the firebox lining, extinguishing the fire. Internal combustion engine generators will stall if pressures of long duration are permitted to enter the exhaust stack. The maximum or minimum blast pressures which will cause such stalling are not specific and engine manufacturers hesitate to specify any limits. At the present time we are considering the installation of a blast-activated poppet valve in the end of the exhaust stack. During the time that the poppet valve is closed by blast, an auxiliary valve, located down in the power plant, will be automatically opened by the back pressure in the stack. In the opened position the valve will permit the engine exhaust gases to be diverted into the access tunnel inside of the blast doors. This scheme requires considerable space in order that concentrations of combustion gases are kept well within the threshold limits for continuous exposure. The estimated time that such diversion is sustained is 10 seconds, after which the poppet valve will open by spring action, permitting normal operation. I must emphasize that such a system requires considerable space in which to exhaust the gases and cannot be used as a general cure-all. This problem is receiving continuous study and may be solved in the near future by the use of nuclear power plants.

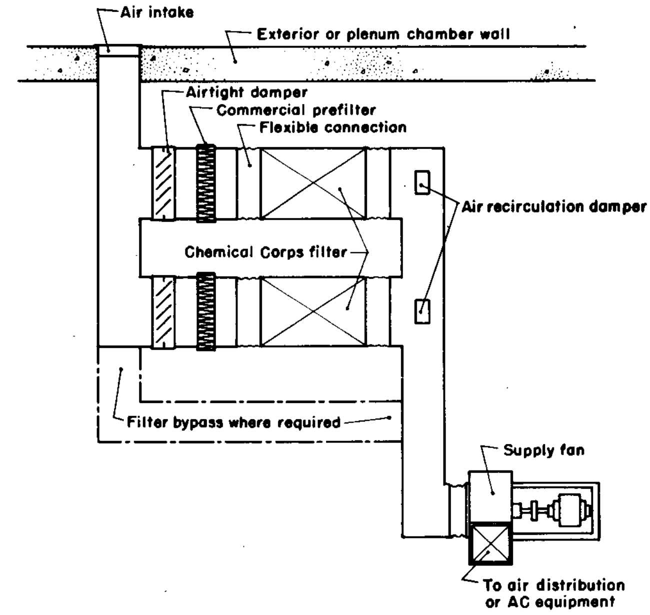
In summary, collective protection against airborne contaminants is accomplished by the installation of special filter units for ventilation air, the pressurization of structures, personnel decontamination facilities for limited ingress, the isolation or separation of combustion air from ventilation air, the provision of blast closure valves which will be activated by the attacking weapon, and last but far from least, the inclusion of an operating procedure for the equipment installed. I might add in closing that all of these details are covered in considerable detail in the Engineering Manual entitled <u>Collective Protection Against Chemical, Biological and Radiological Warfare Agents</u>, EM 1110-345-461.

DISCUSSION

MR. HARRY R. BIEDERMAN (Lockheed Aircraft Corporation, Burbank, California): I was wondering if the combustion air has to be filtered as thoroughly as the breathing air and whether there has been a consideration of unfiltered air.

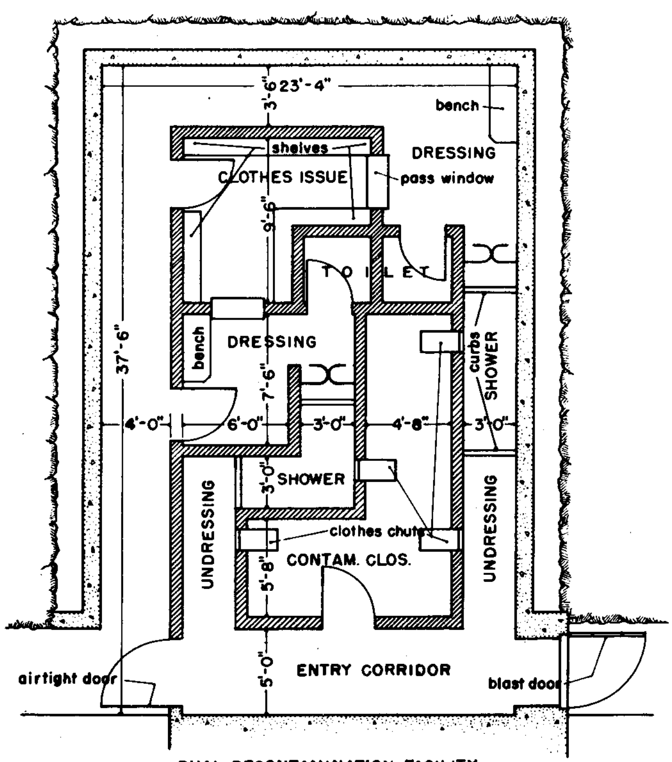
MR. KIRKPATRICK: It does not have to be filtered. There is no evidence that the contamination will be harmful because it will go out with the hot gases.

One difficulty is that if we cannot isolate combustion air from the rest of the installation, the intake pipes may be a source of contamination because of deposits in the intake system. Filters in the intakes might collect contaminants and perhaps become a hazard. However, if they are not in occupied areas this is not too serious. They can be monitored, and it is not necessary for personnel to remain in these areas very long.



FILTER UNIT INSTALLATION

Fig. 1



DUAL DECONTAMINATION FACILITY
(UNDERGROUND INSTALLATION)

Fig. 2

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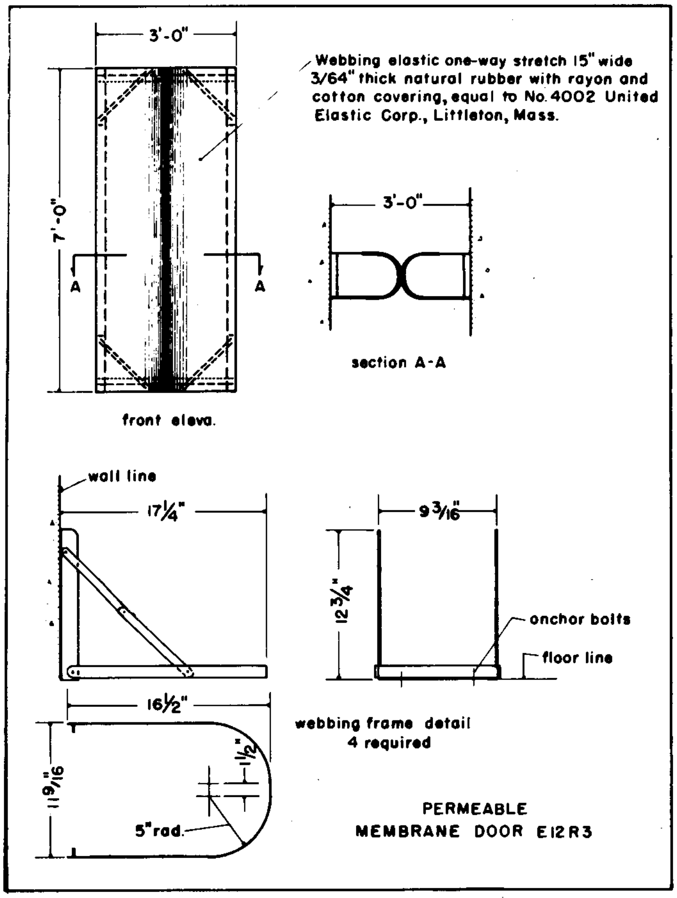


Fig. 3

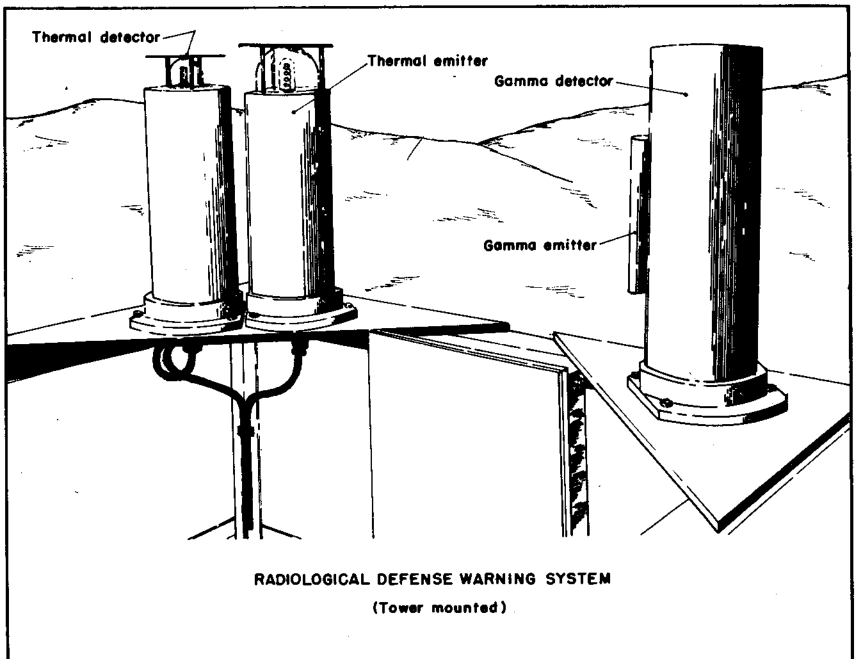


Fig. 4

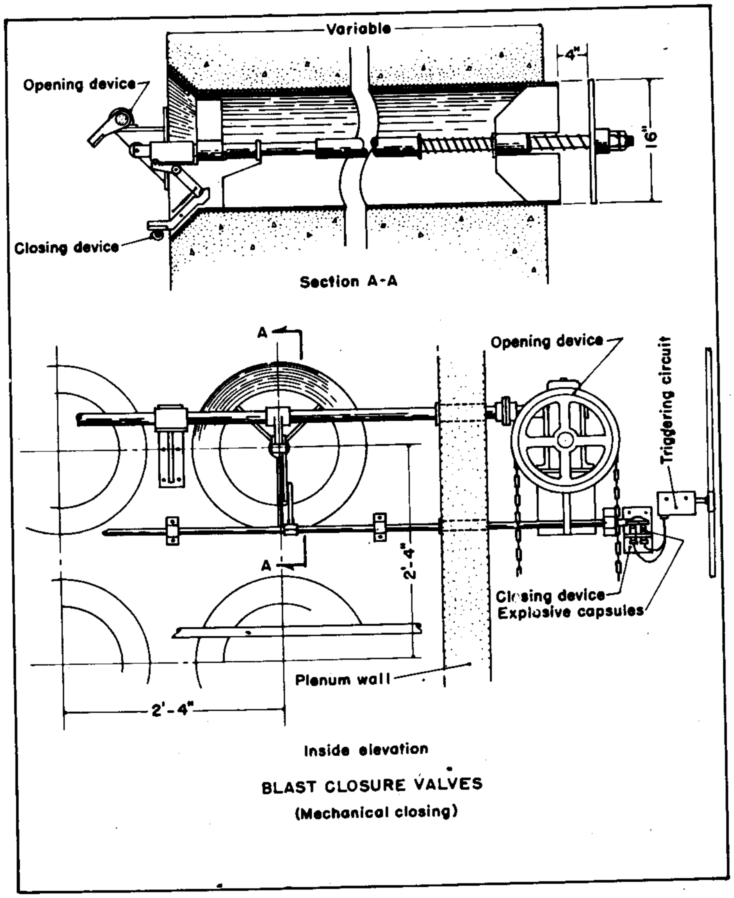

Variable
Opening device
Closing device
4"
16"
Section A-A
A
Opening device
Triggering circuit
2'-4"
Closing device
Explosive capsules
Plenum wall
2'-4"
Inside elevation
BLAST CLOSURE VALVES
(Mechanical closing)

Fig. 5

THE COOLING PROBLEMS OF CHEMICAL AND NUCLEAR POWER PLANT APPLICATION TO A CLASS OF LARGE SHELTERS

W. R. Elswick
The RAND Corporation

This paper is an exploratory discussion of electrical power supplies for large shelters and the associated problems of air breathing and heat removal. The shelter application is considered for a hypothetical military command center or early warning center.

MODEL OF HYPOTHETICAL SHELTER

I've chosen a hypothetical shelter with a rather large power requirement, 8000 kw, which is assumed to be very "hard" by burying it in hundreds of feet of granite or other massive structure. During peacetime operation the 8000 kw power requirement can perhaps be supplied by a nearby municipal or industrial source. However, during wartime operation or a "button-up" period of from, say, 5 to 30 days the shelter must be self-contained. At most the shelter can depend on the outside world for combustion air and cooling air and possibly a body of water as a heat sink.

The heat removal of 8000 kw in peacetime is relatively easy even though the heat is low-grade at low temperature. We can circulate outside air, or impose a water-circulating system between a heat sink--such as a pond or lake with cooling tower or the atmosphere outside--and an air-water heat exchanger within the shelter. The problem thus becomes one of the emergency standby powerplant (or a full time powerplant) and its accessories located within the shelter and a hardened facility to handle the 8000 kw heat load.

HEAT LOADS WHICH MUST BE CONSIDERED

The heat loads which we must consider are:

1. The 8000 kw electrical load dissipated by electronics, lighting and refrigeration equipment.

2. The heat generated by human metabolism--the heat rate based on 1000 people at 150 watts each (3000 kcal/day) is 150 kw.

3. The heat load due to power generation:

   a. The stack gases resulting from combustion products (not applicable to the nuclear powerplant).

   b. The heat load resulting from thermodynamic cycle inefficiencies (this would not be applicable to the case of the gas turbine powerplant).

   c. The heat resulting from the mechanical inefficiencies, generator, pumps, and other accessories.

   d. The heat load resulting from those accessories peculiar to the reflector, moderator and shield cooling loads.

   e. The heat resulting from the heat transfer equipment, pumps, fans, motors, fluid friction, etc.

DURATION AND LOCATION OF OPERATION

For short periods of emergency operation of a few days or so we might store our heat sink underground in or near the shelter in the form of ice and depend only on the atmosphere for combustion if we employ a fuel-air combustion cycle for our powerplant. For longer periods of operation and due to economic considerations, we must look elsewhere for a heat sink. We must tap a heat sink such as the atmosphere, a lake or stream on the surface, or an underground stream. As an example if we chose a nuclear-steam turbine system we would no doubt choose an underground stream or the like rather than the atmosphere as a heat sink. The stream, of course, must have a great enough water flow to handle the entire heat load with an acceptable water temperature rise.

KINDS OF HEAT

For heat transfer and thermodynamic purposes heat comes in different kinds. The most difficult kind of heat to remove is that at low temperature such as the warmed air from electronic equipment, lighting, and human respiration. The coolant is usually at or quite close to the desired temperature level. Here it is sometimes necessary to impose a refrigeration cycle in the cooling system, as the sink and allowable apparatus environment temperature levels are so close. Also, it must be remembered that the operation of a refrigeration cycle introduces some additional heat through the net work done and the inefficiencies present in all systems.

The next most difficult kind (temperature level) of heat arises from such things as powerplant lubrication oil cooling, powerplant radiator cooling, nuclear reactor shield cooling, steam powerplant turbine condenser cooling, and the cooling of electric generators, electric motors and other accessories.

The next temperature level more or less takes care of itself. It is the rejection of the excess heat in the combustion gas along with the products of combustion.

SPECIFIC TYPES OF POWERPLANTS

One of the most common is the fuel oil-fired steam station with a steam turbine driving a generator. We can use fuel oil, as it is easily handled and stored and can be used as a heat sink. Modern steam stations are probably the most efficient type of powerplants available. The major sources of heat which must be removed are: 1) the combustion exhaust gas which is an easy thermal problem as it must be vented to the outside of the shelter; 2) the condenser heat which is at low temperature and of course difficult to remove to the outside or extremely expensive to store within the shelter; 3) the heat from the accessories, which is also low grade heat.

The nuclear-fired boiler steam turbine powerplant, of course, does not have a combustion air requirement. However, it does have the other thermal load requirements of the conventional oil-fired powerplant plus an additional reactor shield and moderator cooling requirement.

Another well known powerplant is the Diesel-generator. Basically the thermal requirements are somewhat less severe than the oil-fired steam turbine and the nuclear-steam turbine powerplants. The Diesel engine does require combustion air but has the advantage of having the combustion air as its thermodynamic working fluid; hence, is able to eject a greater percentage of its cycle heat in the exhaust gas. Even though it is an efficient powerplant it still must dissipate about one-third of its heat to the cooling medium via an engine radiator. Additionally, there are lube oil and accessory cooling requirements.

The open-gas turbine powerplant has some very interesting properties plus one relatively undesirable feature. It essentially consists of a compressor, combustion chamber, gas turbine and generator. It does not necessarily have the severe additional cooling requirements of the other systems unless high thermal efficiency is demanded. Since the gas turbine powerplant operates at very high air-fuel ratios (around 100 to 1) there is an abundance of air which is available for the cooling of accessories as well as other items.

There is another interesting possibility that should not be overlooked since a refrigeration requirement for electronics might exist. High-pressure air at a pressure of 6 atmospheres or greater can be bled from the compressor, intercooled, and expanded through a small turbine to produce refrigerated air at relatively good efficiency. The undesirable feature is the great thirst for air, creating a problem in hardening the power plant. The efficiency is relatively low; however, fuel-oil is not expensive.

COMPARISON OF POWER PLANT PROPERTIES

Table 1 presents some of the properties of the previously described powerplants. Specifically, estimates of the thermal efficiencies, air and fuel rates, and cooling load requirements are included for purposes of comparison.

Several other types of prime movers are possible such as the mercury turbine cycle, the piston steam engine, the internal combustion Otto cycle, as well as hybrids of these. However, the four types represented in Table 1 are not only representative but are probably the logical contenders for the proposed application.

CHOICE OF FUEL

In addition to a possible nuclear source there are several conventional and readily-available fuels which might be considered. Along with fuel oil one might consider coal, gas, and other hydrocarbon fuels which might be

Table 1

COMPARATIVE PROPERTIES OF 8000 KW ELECTRIC POWERPLANTS

Powerplant Type and Efficiency	Combustion Air	Total Heat Load	Heat Load	Fuel Oil Rate	Cooling Water or	Cooling Air
Oil Fuel Steam Turbine 35%	100,000 lbs/hr 1,350,000 $\frac{ft^3}{hr}$	22,900 kw	8000 kw Condenser 6850 kw Stack	4200 lbs/hr 56.0 $\frac{gal}{hr}$	780,000 lbs/hr (100° F Rise) 94,000 $\frac{gal}{hr}$	3.3×10^6 lbs/hr (100° F Rise) 44×10^6 $\frac{ft^3}{hr}$
Nuclear Fuel Steam Turbine 35%	---	22,900 kw	12,600 kw Condenser 2300 kw Shield	---	860,000 lbs/hr (100° F Rise) 104,000 $\frac{gal}{hr}$	3.6×10^6 lbs/hr (100° F Rise) 48×10^6 $\frac{ft^3}{hr}$
Diesel Electric 33% 0.55 $\frac{lbs/fuel}{kw\text{-}hr}$	129,000 lbs/hr 1,800,000 $\frac{ft^3}{hr}$	24,000 kw	8000 kw Radiator 8000 kw Stack	4400 lbs/hr 590 $\frac{gal}{hr}$	310,000 lbs/hr (100° F Rise) 37,300 $\frac{gal}{hr}$	1,280,000 lbs/hr (100° F Rise) 18×10^6 $\frac{ft^3}{hr}$
Open Gas Turbine 25%	560,000 lbs/hr 7,600,000 $\frac{ft^3}{hr}$	32,000 kw	24,000 Stack	5600 lbs/hr 750 $\frac{gal}{hr}$	---	---

available to the shelter. Diesel oil, of course, is a specific requirement for the Diesel powerplant. I have specifically considered fuel oil as it is more easily handled and stored than coal, more easily sheltered than gas and offers the possibility of being used as a low temperature heat sink within the shelter. Fuel oil has a specific heat one-half that of water. (Natural gas has a higher specific heat although its low density probably excludes it from consideration as a fuel for a very high overpressure shelter.)

MODE OF OPERATION AND SITE LOCATION

The mode of operation of the shelter determines to some extent the site location and the choice of powerplant type.

EMERGENCY OPERATION ONLY

If the shelter site is close to a suitable source of power such as a city, an industry with excess power or a TVA dam powerplant, presumably the shelter would require only a standby type of powerplant. In such a standby plant thermal efficiency might not be of great importance. However, a short start-up time would, no doubt, be highly desirable.

CONTINUOUS OPERATION

If the powerplant were isolated from a suitable civilian power source and the sheltered powerplant must run continuously, then a fuel source must be available, the powerplant should operate efficiently, and the necessary cooling requirement for continuous operation must be met. In the case of the steam turbine plant, the nuclear steam turbine plant, and the Diesel engine plant large impressive cooling systems must be provided.

In fact an underground stream would be highly desirable as the heat sink, thus imposing an additional and perhaps severe constraint on site selection. Presumably if one can solve the combustion air problems in shelter design for fuel-burning powerplants such as the steam and Diesel powerplants he can also accomplish the cooling requirement with atmospheric air if necessary. However, it should be noted (see Table 1) that air cooling of such powerplants requires about 10 times the air-handling capacity over that of the combustion air alone. Air-cooling the nuclear powerplant also requires a similar air-handling capacity.

The gas turbine requires only about six times the combustion air of the other hydrocarbon powerplants; however, its efficiency is somewhat lower, indicating a 13 to 15 percent greater fuel requirement.

Start-up time would, of course, be of little concern if the powerplant were to operate continuously.

AIR SHAFT SIZES AND HEAT TRANSFER AREA

An air-cooled powerplant even as small as 8000 kw requires a prodigious quantity of air if that air must be piped and pumped a thousand feet or so underground, heated, and piped 1000 ft back to the atmosphere. Table 2 contains an estimate of the required flow rates, duct sizes, and pumping power to bring the air in and then to pump it out. No attempt had been made to estimate the power required to pump the air through heat exchangers which transfer the heat from the electronics to the cooling airstream. The power requirement would, however, be appreciable.

For this type of heat transfer where air is the transfer fluid a heat transfer coefficient of 2 to 5 Btu/hr ft^2/^{o}F is usually realized in industry. This corresponds to about 0.001 kw/ft^2/^{o}F. Therefore a load of

Table 2

COOLING AIR DUCT SIZES AND PUMPING POWER
(Air Velocity in Duct 100 ft/sec)

Powerplant Type	8000 kw Internal Load	Powerplant Load Excluding Internal Load	Powerplant Load Pumping Power
Oil Fuel Steam	(50° F Rise) 2.5×10^6 lb/hr 34×10^6 ft^3/hr Diameter = 11 ft *1300 kw power	(100° F Rise) 2.3×10^6 lbs/hr 31.5×10^6 ft^3/hr Diameter = 10.5 ft	1200 kw
Nuclear-Fired Steam	Same *pumping power is 1300 kw	2.7×10^6 lbs air/hr 37×10^6 ft^3/hr Diameter = 11.5 ft	1100 kw
Diesel Electric	Same *pumping power is 1300 kw	Same as above	1100 kw
Open Gas Turbine	Same *pumping power is 1300 kw	560,000 lbs/hr 7.6×10^6 ft^3/hr (STP) D_{in} = 5 ft D_{out} = 7 ft	245 kw

*The 1300 kw pumping power would be that power required to cool the internal 8000 kw electrical load by outside air if we were to heat the air by 50° Fahrenheit.

8000 kw would correspond to 8,000,000 ft^2/$^\circ$F of heat transfer surface. Even if we were able to use the full 50° F temperature difference the required heat transfer area would be 160,000 ft^2, almost four acres.

The combustion air requirements are rather modest by comparison. Table 3 is a breakdown of my estimate of those requirements showing the duct sizes and the pumping power for the various types of powerplants. It was assumed that the "chimney effect" of the heated gas offered no net pumping effect. That is, chimney pumping tended only to offset the pressure drop due to blast trap devices which must be in the air lines.

SUMMARY

The net electrical load requirement designed into the shelter is appreciably magnified by the time the powerplant meets all the requirements of refrigeration, heat transfer, and fluid flow.

Using the atmosphere as a heat sink for a shelter with a net electrical load of around 8000 kw does not appear to be particularly desirable.

A built-in heat sink such as an underground stream of sufficient capacity to carry away the heat and yet suffer only a low temperature rise is most desirable. An alternative heat sink might be a lake in the vicinity of the shelter which could be tapped from the bottom.

The combustion air requirement does not appear to be at all unreasonable to supply. The air ducts are not particularly large for the 8000 kw net electrical powerplants which have been considered.

The generation of heat at low temperatures such as in electronics, human respiration, and lighting creates the most difficult heat transfer and refrigeration problems within a shelter unless that shelter happens to

Table 3

COMBUSTION AIR DUCT SIZES AND PUMPING POWER

Powerplant	Combustion Air Flow Rate	Duct Sizes Based on 100 ft per sec Velocity IN	OUT	Combustion Air Pumping Power
Oil Fuel Steam Turbine	100,000 lbs/hr 1,350,000 $\frac{ft^3}{hr}$	20 inches diameter	35 inches diameter	224 kw (300 hp)
Nuclear Fired Steam Turbine	---	---	---	---
Diesel Electric	129,000 lbs/hr 1,800,000 $\frac{ft^3}{hr}$	23 inches	41 inches	250 kw (330 hp)
Open Gas Turbine	560,000 lbs/hr 7,600,000 $\frac{ft^3}{hr}$	47 inches	80 inches	520 kw (700 hp)

have a large heat sink. The atmosphere is difficult to use and an ocean or lake is usually not present.

Every effort should be exercised to cut the net electrical power requirement to a minimum. Transistorized electronics designed with cooling in mind should be employed wherever possible, lighting should be at a minimum and should be of the more efficient gas discharge type such as the fluorescents.

The nuclear power source at best only eliminates the combustion air requirements. Even assuming the same over-all efficiency as the oil-fired heat source, a similar heat transfer and cooling problem exists.

INTRODUCTION TO SESSION ON NEW CONSTRUCTION METHODS AND EQUIPMENT

R. L. Loofbourow
Consulting Engineer

In this session we have to consider a broad subject. The great diversity in the physical properties of rocks was nicely stated by Mr. Wilbur Duvall in his paper, "Design of Underground Openings for Protection." In addition, there is quite as great a diversity in what may be called the physical condition of rocks -- that is the type, severity, spacing and orientation of defects such as bedding, fracturing, various forms of chemical alteration, the presence of abnormal stress, and the occurrence of rock fluids such as water and, rarely, oil and gas. Other conditions such as topography and the purpose of the work may establish various requirements as to the nature of entrances and the shape of excavations.

To cope with these diversities we may choose from a broad range of methods and equipment. Since cost is very much an object, it is especially desirable that sites be selected so as to facilitate construction and also that the conditions be determined as accurately as possible, to permit best design and selection of efficient methods and equipment.

We are concerned not only with the cost of excavating but with the cost of making stable excavations. Though we understand that the point is questioned, in this session we are guided by the assumption that for the type of work contemplated and under reasonably favorable conditions of rock stress, an excavation of a given shape, size and orientation, at a given spot will be most stable if its roof and walls are comparatively smooth and unshattered. This is one of the reasons for our interest in boring methods.

Some boring methods promise to become most appropriate for establishing entrances through rock which would be slow and difficult by conventional mining. The possibility of boring curved holes may aid in the dispersal of entrances.

Boreholes of comparatively small diameter may be highly useful for all manner of conduits from the surface to underground and between separate underground excavations.

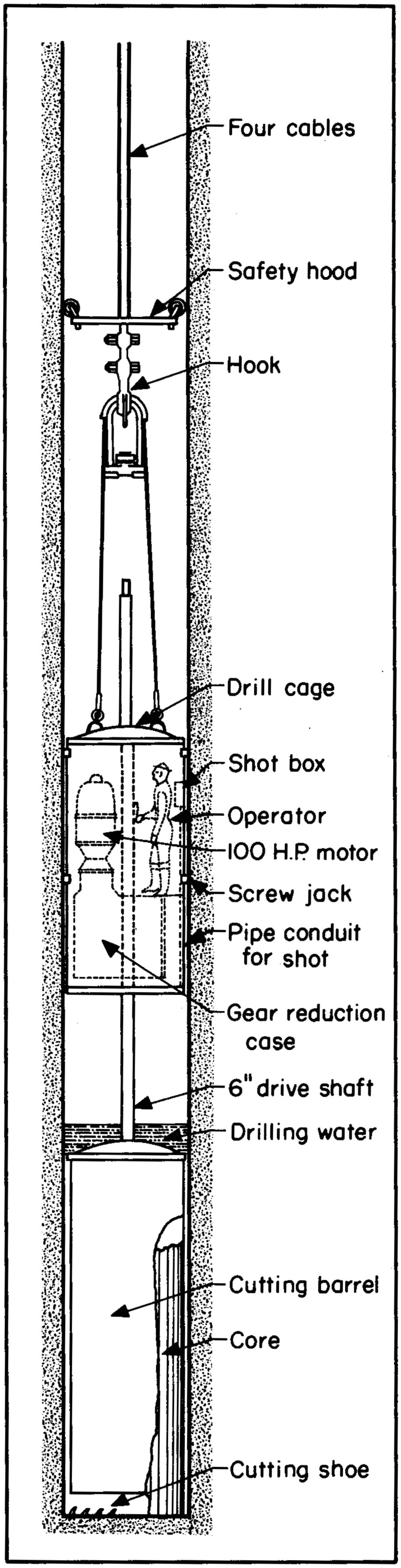
To achieve a certain degree of completeness with respect to boring, several types of equipment, not otherwise described, will be mentioned here.

Shot drills, illustrated by Fig. 1, use chilled steel shot fed under a special cutting shoe to abrade rock of any hardness, cutting a core which must be detached and removed from the hole piecemeal. Cutting may be slow and progress is subject to further delays for core removal. The type of drill illustrated puts motive power and the drill operator in the hole. Another type drills from the surface using drill pipe to rotate the core barrel and shoe.

A slabbing saw or a chain saw with hard metal teeth developed for use in salt and coal has been used ingeniously in a periphery saw for cutting 33-ft tunnels in chalk. It is illustrated in Fig. 2. In operation the saw is advanced into the face, then caused to traverse around the periphery of the tunnel. A few blastholes were used to break up the "core." This is not a continuous operation. Application of the saw would appear to be limited to soft rock such as clay-shale, salt and chalk. Compare in this photograph the lower part of the vertical wall, which was cut by a slabbing saw, with the line-drilled center. Figure 3 shows the periphery saw just after holing through one of the same tunnels. Note that such a saw could be modified to cut any prismatic shape.

A number of continuous mining machines are regularly manufactured and used in coal and salt mining, producing these materials in volume at low labor costs. They could be useful for making excavations for single-story structures in salt, for instance. Power augers made by several firms for surface and underground use can dig holes in earth, clay, coal and salt with extraordinary rapidity, vertically or at any angle, to diameters of 5 ft or in favorable circumstances a few more feet, and to depths of 50 ft to several hundred feet, depending on the size of the equipment, the diameter and angle.

FIGURES



Newsom type core drilling equipment for 5.5 foot diameter shafts using chilled steel shot as abrasive

Mining World, September, 1957

Fig. 1

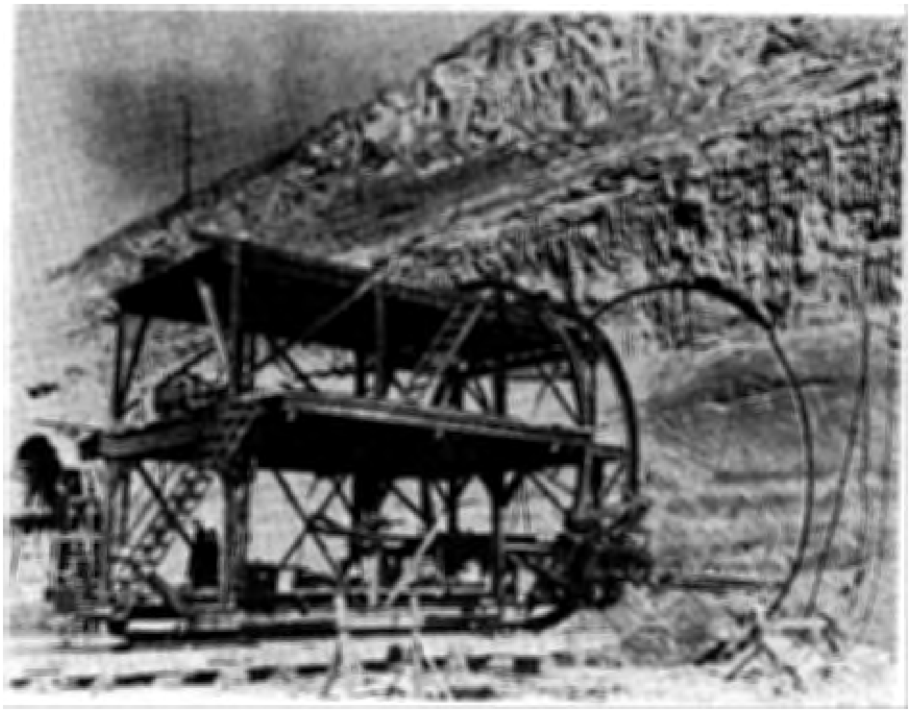


Fig. 2

Fig. 3

ADAPTATION OF OIL WELL DRILLING TECHNIQUES

Gene Graham
Graham and Klassen Drilling Company, Inc.

Just one century ago, in August of 1859, Colonel Edward Drake drilled the world's first oil well--a hole 69-1/2 ft deep. This was a post-hole by today's standard but a most important contribution to the progress of the last hundred years. The drilling and completion of the Drake well was the key that unlocked the liquid riches of this earth--the oceans of oil on which our entire industrial economy floats.

Today's oil wells are basically the same as Drake's well in that they are holes drilled into the ground to the depth where oil has accumulated. While the average well's depth has now reached 5000 ft, a dozen or so wells have passed 20,000 ft, and one, drilled in Oklahoma last year, passed 25,000 ft. The string of drill pipe, in its diameter-to-length ratio, would be proportionate to a human hair 5-ft long!

In order to discuss our subject it will be helpful to review briefly how a modern oil well is drilled--from the driving of the surveyor's stake to the completion of the well.

First, a graded site with a means of access is needed. A sump hole and a "cellar" are constructed on the site, and a conductor pipe is cemented in place. The sump is merely a hole large enough to collect waste fluids. The cellar is actually the beginning of the well. The conductor pipe prevents the mud flow from washing out the rig's foundation. When these are ready, it is time to move in the drilling equipment.

The equipment, commonly called a "rig," consists of:

1. A hoist, referred to as the "drawworks," which is the machine used

to raise and lower the drilling string;

2. A derrick or mast, the structure which looks like a tower and in which drill pipe is racked on trips;

3. Pumps, used for circulating the drilling fluid;

4. Prime movers to power the drawworks and pumps;

5. The drill string, which is actually a pipe line--as much pipe in 30-ft lengths as is needed to drill the hole to its total depth, and

6. Many auxiliary and accessory items.

A schematic drawing of a complete oil well drilling rig is shown in Fig. 1.

All of these items are bulky and heavy; they are designed for rough service, with road restrictions the only critical criterion. California laws prohibit loads over 8-ft wide on the highways without special permits, and when pieces of equipment are larger than this limit, they are divided for road transport.

Figure 2 illustrates a typical drawworks - power package. Portions of shallow-drilling rigs may be permanently mounted on heavy-duty trailers. Such mobile units are usually limited in their hoisting and pipe handling capacities to about 4000-ft wells.

Figure 3 is a photograph of a mobile trailer-mounted "blitz" rig ready for moving.

A location road from established highways to the well site necessarily must be competent to handle loaded trucks; its curves must be gentle enough to permit passage of loads of up to 50 ft in length. In mountainous areas the cost of access roads may become prohibitive.

Recently in Guatemala a big drilling outfit, complete with four hundred 30-ft joints of drill pipe, was transported by helicopter and airplane from a seaport to an inaccessible jungle location. The total weight of this drilling rig and drill string was in excess of 600 tons. While it is true that a few of the major components of this outfit were especially designed for the project, most pieces were standard design. The possibilities of this mode of rig transport are now available to us.

Figure 4 is a photograph of an oil well drilling rig on location, and Figure 5 is a schematic drawing of a "blitz" rig in operation.

When the equipment is transported to the well site, it is assembled in correct relationship to the exact surface location of the proposed well bore and the well is "spudded in"--an oilfield term meaning the beginning of the drilling operation.

State and Federal laws require that the fresh waters directly beneath the surface of the ground be protected from contamination promptly after spudding the well. Oilfield practice calls for a means to protect property against blowouts. For these reasons a string of large-diameter pipe, usually 11-3/4 in. OD, is cemented to a depth of as much as 1000 ft. The installing and cementing of this pipe in the hole protects the water strata and provides an anchor or foundation upon which the blowout control valves are installed in the cellar on the surface.

After these steps are accomplished, generally within two days after spudding, the real drilling job begins. From this point on the "technique" of drilling must be employed.

The rotary method of drilling requires that a circulating fluid, called "mud," be pumped constantly down through the drill string while drilling, through the drilling bit, and back up the surface in the annulus between the hole and the drill string. The fluid serves four main purposes: (1) it carries the cuttings from the bottom to the surface where they are separated from the fluid; (2) it cools and lubricates the drilling bit; (3) it holds back fluids and gases which would enter the well bore if the hydrostatic head of the fluid were not present; and (4) it keeps certain types of formations from sloughing and "heaving" into the bore. The subject of "mud" is a complicated one; it is actually a problem in chemistry by which weight, viscosity, p^H factor, and other properties are controlled.

The drill string is the tool which actuates the drilling bit. It is hollow, to permit the passage of the mud, and has, at its lower end, heavy thick-walled steel tubes called drill collars. While there is much controversy on the subject, it is generally agreed that the drill pipe should be run in tension, using the drill collars as the weight on the bit. Each 30-ft joint of drill pipe is equipped with a "tool joint" having coarse threads on a large taper for quick make-up and break-out.

Figure 6 shows a typical drill string tool joint. Today practically all rotary-drilled holes are drilled with roller cutter bits. Over the years the tri-cone design has emerged as the most economical tool. Such a bit has three conical cutters with teeth machined on the cones and tungsten carbide applied to the cutting surface. The pattern of the teeth is varied--larger and more widely spaced for softer formations, smaller and closely spaced for harder formations.

Figure 7 shows a typical roller cutter drilling bit.

The drag-type drilling bit has practically disappeared from use although it still has some application in soft sandy formations.

Figure 8 shows a typical drag-type drilling bit.

With a fresh bit run into the hole on the drill string to a point a few feet off bottom, circulation of the mud is started, rotation of the string is begun, and the driller lowers the string until his weight indicator tells him he is on bottom. Generally, depending on the formation being penetrated, the rotary speed will be from 50 to 200 rpm--fast for soft formations and slow for hard strata. The cutters on the bit rotate on their axes, making approximately four revolutions for each turn of the drill string. The teeth of the cutters chip away on the bottom, being pressed into the formation by the weight of the drill collars. It is not uncommon to apply 5000 lb of weight per inch of bit diameter. Here again as a general rule heavy weight and slow rotation are used in hard formations; lighter weight goes with faster rotation in softer formations.

Experiments are now in progress on several novel methods which may change our techniques. The turbine bit is in actual use, a sonic tool is in the experimental stage, and flame or jet cutting has been tried.

The rate of circulation of the mud is extremely important to effective drilling. In order to assure that the bit drills on new formation, it is necessary to flush the cuttings away and carry them to the surface. A rising velocity in the annulus of 3 ft/sec is considered the minimum for this purpose.

Flow of the mud through the orifices (jets) of the bit flushes the cuttings away from the bottom so the bit may always have new formation to work on. Mud velocity through the orifices should be in the range of 200 to 400 ft/sec.

Figure 9 is a schematic drawing of the fluid flow through a roller cutter drilling bit.

In oil well drilling a "straight" hole is one which does not deviate more than 3 deg from vertical. Three degrees, however, permits a horizontal deflection of 50 ft per 1000 ft of depth and, in many instances, it is necessary to control a hole more closely. Conversely, at times it is essential to direct a well several thousand feet from the vertical projection of the surface location.

Control of well bores, whether to maintain an extremely vertical hole, to achieve a considerable horizontal "kick," or to alter the direction, is done by the use of a whipstock. This tool is actually nothing more than a steel wedge which acts to force the drilling bit away from the present course of the hole.

Figure 10 illustrates the use of a whipstock.

The whipstock is fixed to the drill string and a drilling bit, smaller than that which drilled the hole to this point, is made up on the bottom of the drill string. The whipstock is lowered to the bottom of the hole and oriented to face in the desired direction. Drilling is begun, with the wedge-like whipstock forcing the bit to deviate from the former course. After drilling 10 to 15 ft, drilling is suspended, the drill string is raised to engage the collar of the whipstock, and the tools are removed from the hole. Normal drilling is resumed in the whipstocked hole.

Generally speaking, a whipstock run will cause a 3 deg change in deviation from vertical. As the angle of a hole increases, the effect of a whipstock run is greater in most formations.

After a hole is "kicked off" it is customary to use stabilizing tools in the drill string to continue building angle or maintain a desired angle. The position of these tools near the lower end of the drill string, in relation to the bit, determines the action of the string. While there are several different types of stabilizing tools, each is basically a short section of drill string with "wings" or blades which contact the walls of the hole. The stabilizer is used as a fulcrum to pry the bit up in angle-building operations.

Periodic surveys are made to check the course of the well as drilling progresses. To accomplish this the drill string must contain a section of non-magnetic material, either "K monel" or stainless steel, run directly above the bit, into which a recording type of survey instrument is dropped from the surface. From these "shots," which indicate angle of deviation and direction, the well's course is plotted.

Figure 11 shows a typical directional drilling well completion report.

When the hole is drilled to the desired depth, the drill string is removed from the hole for the last time, the hole is cased, and the casing is cemented in place. After a period, usually not less than 24 hours or more than 48 hours, completion operations are undertaken.

Figure 12 shows the four steps in the drilling and completion of an oil well (vertical scale distorted).

It may be helpful, in connection with this very brief resumé of oil well drilling, to recognize the speed, or lack of it, with which wells are drilled. In the soft sands and shales of the San Joaquin Valley it is possible to reach 10,000 ft in 15 days from spudding. This is "snowbank" drilling. On the other hand, 7000 ft of hole in the Sespe shale formations near Ventura require 45 to 50 days.

The cost of operating an oil well drilling rig will vary from $900 per day for shallow drilling with the "blitz" outfits, to $1500 per day for the biggest rigs. These figures are for rig operation only and do not include costs of mud, bits, fuel, crew subsistence, etc. Mud may cost as much as $200 per day, diesel fuel up to $100 per day, and bits about $400 per day.

Up to this point this discussion has been concerned with oil wells--small diameter holes drilled to considerable depths. The problem at hand--the drilling of large-diameter, relatively shallow, directed holes--poses several problems:

1. How to achieve the final hole size?

2. How to circulate the drilling fluid for proper cleaning of the hole?

3. What size and type of drill string is practical?

4. How to direct the hole to the target?

5. How to protect the bore with casing?

To achieve a directed bore 50 to 60 in. in diameter and no more than 2000 to 2500 ft in length will require the drilling of a "pilot hole" of 10 to 12 in. in diameter. In a hole of this size normal "blitz" equipment will suffice. This size hole will also be relatively simple to direct to the desired target. In this pilot hole there will be no more than the usual problems of circulating fluid, maintaining directional control, and drilling with the usual 4-1/2 in. OD drill pipe. The ideal bore will start vertically at the ground surface, build angle at the maximum rate possible, and end up horizontal.

It is generally agreed that an angle buildup of 5 deg per 100 ft is the maximum possible to achieve and still maintain conditions which will permit the use of the 4-1/2 in. drill pipe.

Hole enlargement may be achieved in one of two ways: in steps of 8 to 12 in. per step, or in one operation. Recently, in a mining operation in Arizona, an oil well rig was engaged to drill vertical holes 48 in. in diameter. In this job the program called for a 15-in. hole enlarged to 26 in., then to 36 in., and finally to 48 in. The hole-enlarging tools were roller cutter tools with a pilot.

Figure 13 shows typical roller cutter hole-enlarging tools.

This operation was not too successful because straight-forward oil well techniques were used; no attempt was made to alter the equipment for greater effectiveness. The following modifications should be advantageous to achieve the final hole size.

Ordinary oil field mud pumps do not have the capacity to maintain a 3 ft/sec rising velocity in the annulus of a large hole. One may easily calculate that pumps capable of giving the desired rising velocity in a 12-in. hole will deliver approximately 1/4 of that in a 24-in. hole, and 1/16, or .18 ft/sec, in a 48-in. hole. Such low velocities cannot carry the cuttings away from the bit and up the hole.

The probable answer to this problem is to reverse the flow, circulating down through the annulus and into the hollow drill string. The hazard in this is that reverse circulation tends to apply greater pressure on the formations which may be "broken down," with the result that circulation may be lost into the formations. Corrective measures are available, however, if this occurs.

Assuming that "reverse circulation" will solve the problem of carrying the cuttings up to the surface, the next consideration is the size of the drill string.

Our present 4-1/2-in. OD drill pipe is a well-engineered tool for its intended purpose, but it simply cannot stand up under the extreme stresses imposed by very large diameter drilling tools. The Arizona experience showed that one of two things happened when it was used with the large roller cutter hole-enlargers. Any attempt to make reasonable progress in the hole resulted in twisting off the drill string. Operating under speed and weight conditions which allowed the drill string to remain effective resulted in little or no hole progress.

Obviously a larger diameter drill string is the answer. It is suggested that pipe similar to 9-5/8 in. OD, 53.5-lb (.545-in. wall thickness), Grade P-95 (95,000-psi yield strength) seamless oil well casing with special tool joints will do the job. Our regular 4-1/2 in. OD drill pipe with tool joints costs $6.50 per foot today; the suggested string will probably cost $20.00 per foot.

The heavier, larger diameter drill string will preclude the use of the same hoisting equipment which was adequate for the drilling of the pilot hole. This means that (1) a larger, more capable rig should be used to drill the pilot hole in order to be able to complete the hole-enlarging operation, or (2) the "blitz" equipment used for the pilot hole would have to be moved out and larger equipment moved in for the hole-enlarging operations. Since it is always hazardous to let a hole stand for even the two days required to change rigs, the first course would appear to be the better.

With the circulation proceeding upward through the 9-5/8 in. drill string, modifications of the surface equipment are necessary. The kelly, which is the topmost member of the drill string and which is the driven piece of the string, must be larger than a normal kelly in the same general ratio as the larger drill string is to a normal string. The rotary hose, the flexible connection between standpipe and kelly, must also be similarly larger to handle the flow, but, fortunately, since it is now on the discharge or low-pressure side of the circulating system, it is a low-pressure member. It is probable that the resilient hose could be replaced by a steel pipe-swing joint arrangement which would eliminate the standpipe. The swivel would require an increased flow area throughout.

An alternate method which deserves consideration is to use compressed air as a circulating fluid instead of mud. This method has some advantages;

the principal one is the elimination of the need for large quantities of water, which may be a problem. In using air, the annulus between hole and drill string becomes, in fact, a large receiver tank. Safety laws usually require that the exhaust air be directed through a water spray to drop the cuttings. Obviously if the formations penetrated are not competent to contain the air under pressure, the method has no merit.

With the modified equipment and techniques described, the hole will be achieved, but it cannot be put to any use until it is protected by steel casing. The design of a string of casing is a relatively simple function for oil wells, and the rules used will apply here with one added item for consideration: the bending of the casing to conform to the course of the directed hole. It is probable that steel pipe, whose diameter is 18 in. less than the hole diameter, of proper grade steel and correct cross-sectional area, could be run satisfactorily.

In conclusion, and in summation, it should be understood that the modification of oil well drilling equipment to meet the requirements of producing holes 30 or 60 in. in diameter directed to a target relatively far from the surface location is not a difficult task. Fortunately only a very few of the components of a rig need modification and these are relatively inexpensive pieces. The larger drill string is probably the most expensive acquisition indicated. A "guess-timate" of the cost of the modified equipment, excluding the drill string, is $25,000.

One should not assume that merely making the few modifications mentioned will guarantee success. When a hole is dug into the ground by any method, many variables are encountered which were not apparent at the start. The very nature of drilling, by applying power at the surface to do work hundreds and thousands of feet away from the point of power application, means that highly-skilled personnel is the most important factor in the success of any drilling operation.

The oil well drilling industry is ready to help you solve your problems. It is an industry which thrives on tough jobs and rush schedules.

FIGURES

A SCHEMATIC DRAWING OF A COMPLETE
OIL WELL DRILLING RIG

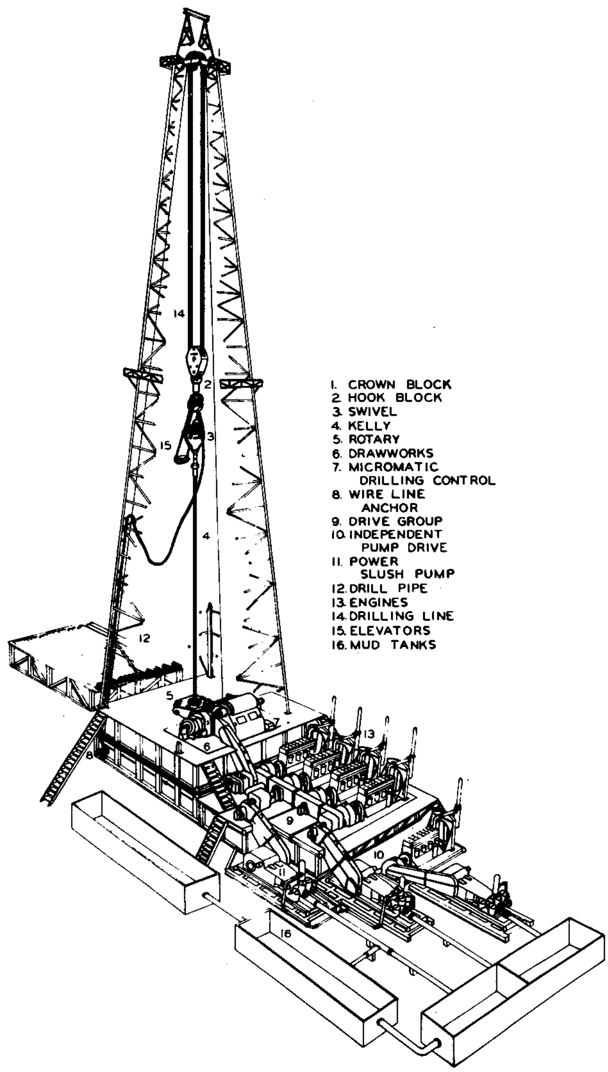


Fig. 1

AN ILLUSTRATION OF A TYPICAL
DRAWWORKS—POWER PACKAGE

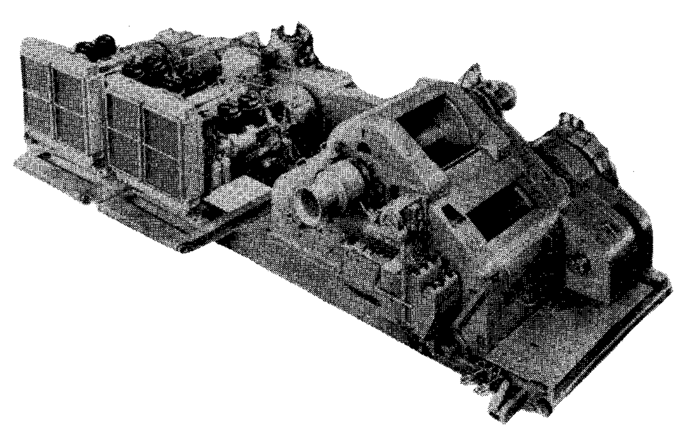


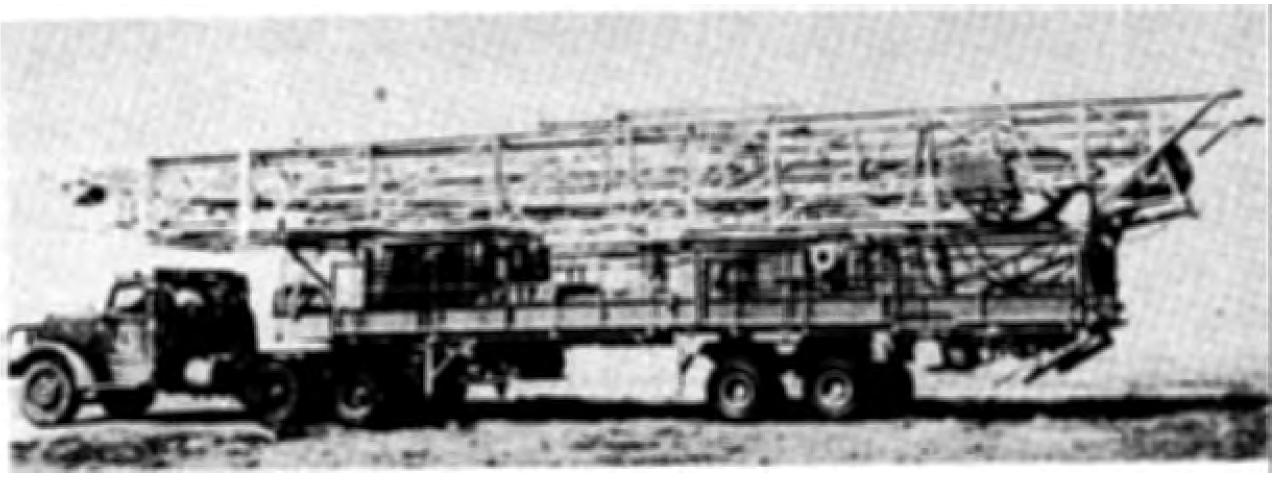
Fig. 2

R-341
3-26-59
604

A PHOTOGRAPH OF A MOBILE TRAILER—MOUNTED RIG READY FOR MOVING

RIG TRAILER:

Overall dimensions (with derrick): Length—61'4"; Height 13'6"; width 8'10". Weights: at kingpin 20,640 lbs.; Gross 71,740 lbs.; at bogie 51,100 lbs. For double drum units add: At kingpin 2,350 lbs.; Gross 6,500 lbs.; at bogie 4,150 lbs. Based on standard single drum unit with two Cummins NHRIS 600 Diesel Engines, hydratarder brake, 102-ft. derrick, and walkways as shown below:



SUBSTRUCTURE TRAILER

Overall roading dimensions: Length—35'0" Height 8'6" Width 8'0". Weights: At bogie 24,000 lbs.; Gross 27,740 lbs.; at kingpin 3,740 lbs. Based on standard unit with 8' high substructure, 17" table, elevated rotary drive, and pipe racks loaded as shown below:

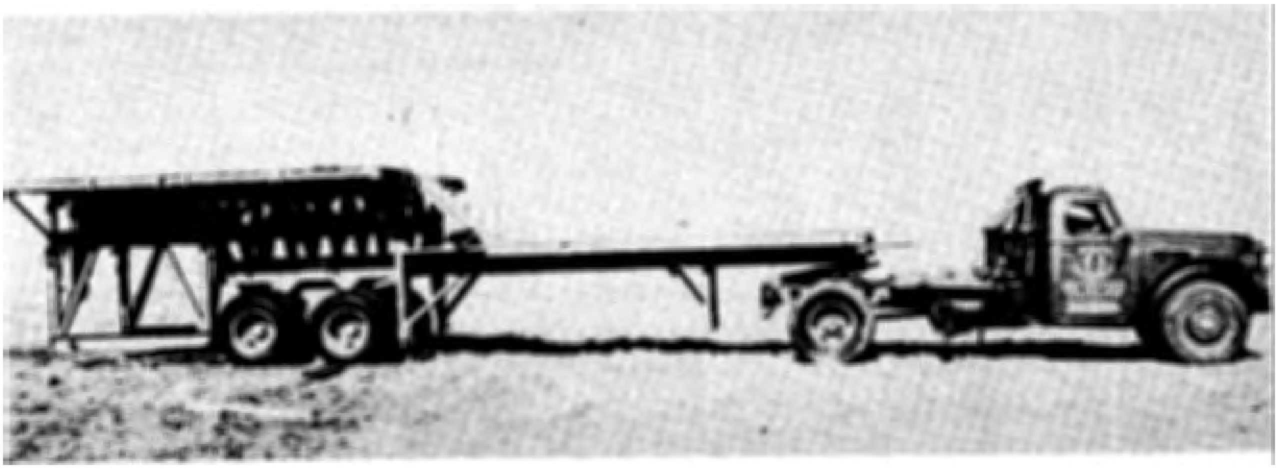


Fig. 3

AN ACTUAL PHOTOGRAPH OF AN OIL WELL DRILLING RIG IN OPERATION

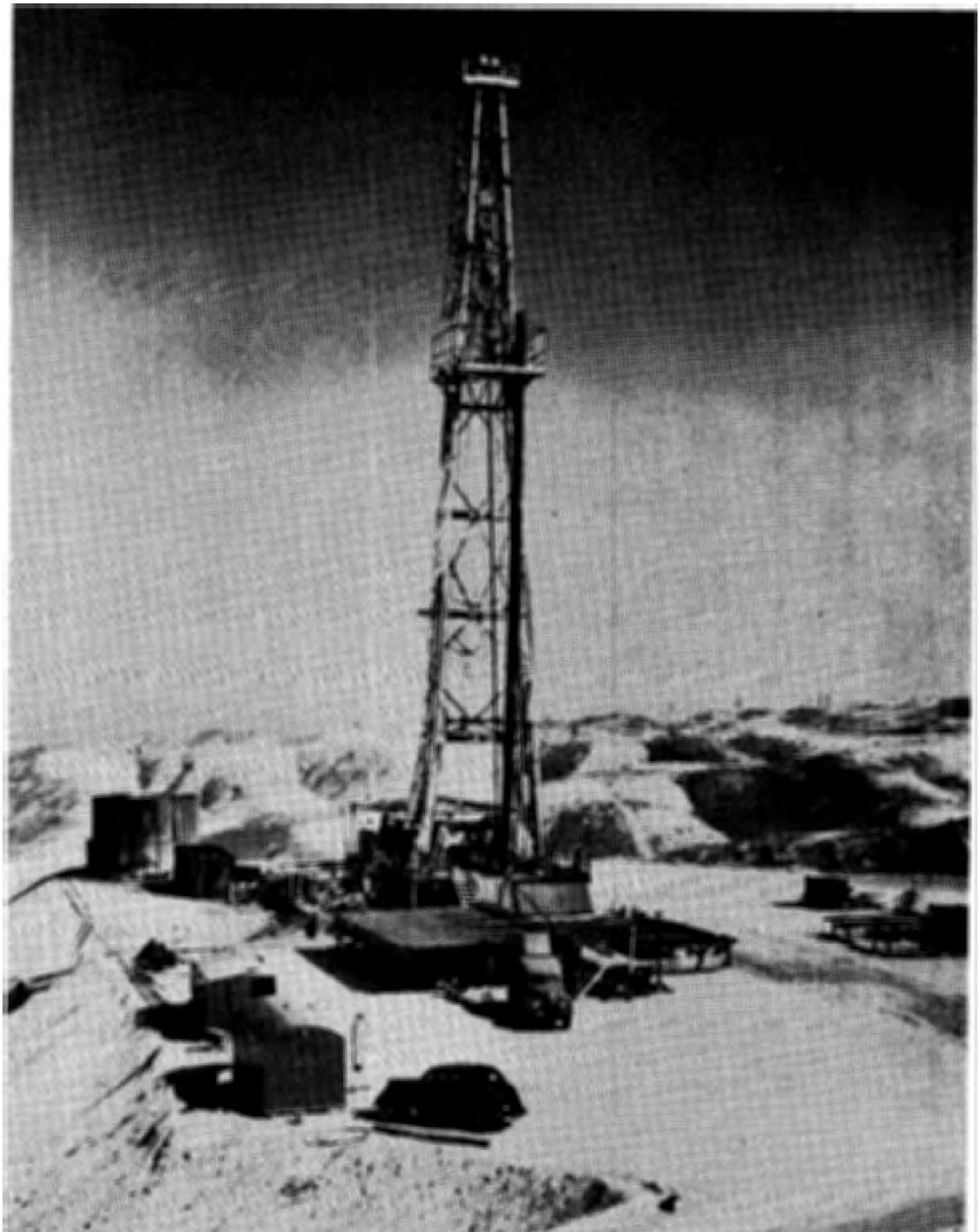
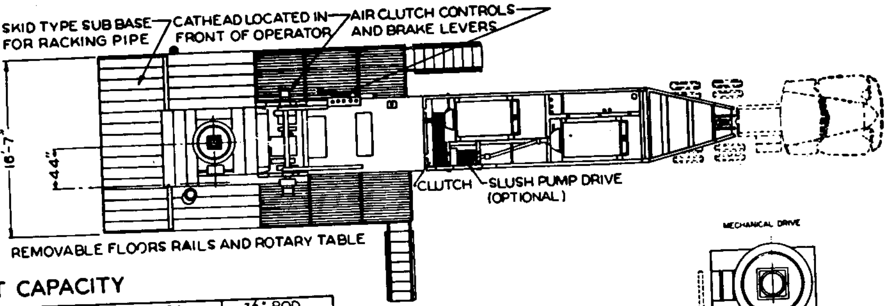


Fig. 4

R-341
3-26-59
606

A SCHEMATIC DRAWING OF A MOBILE TRAILER-MOUNTED RIG

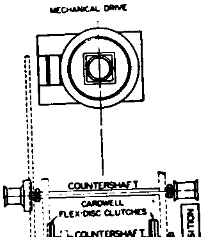


MAST CAPACITY

MAST MODEL	WORKING HOOK LOAD	RACKING CAPACITY IN DOUBLES			7/8" ROD IN TRIPLES
		3" TUBING	2½" E.U. TUBING	4½" X 16.6" D.P.	
200	200,000 LBS	10,000 FT.	12,000 FT	6000 FT	7,000 FT.
201	200,000 LBS	10,000 FT.	12,000 FT.	6000 FT	7,000 FT

DIMENSIONAL DATA

MAST MODEL	A	B	C	D	E	F
200	91'-8"	56'-6¾"	6'-5"	5'-4"	6'-2"	59'-5½"
201	102'-1¾"	55'-9½"	7'-7"	6'-6"	7'-4"	65'-10½"



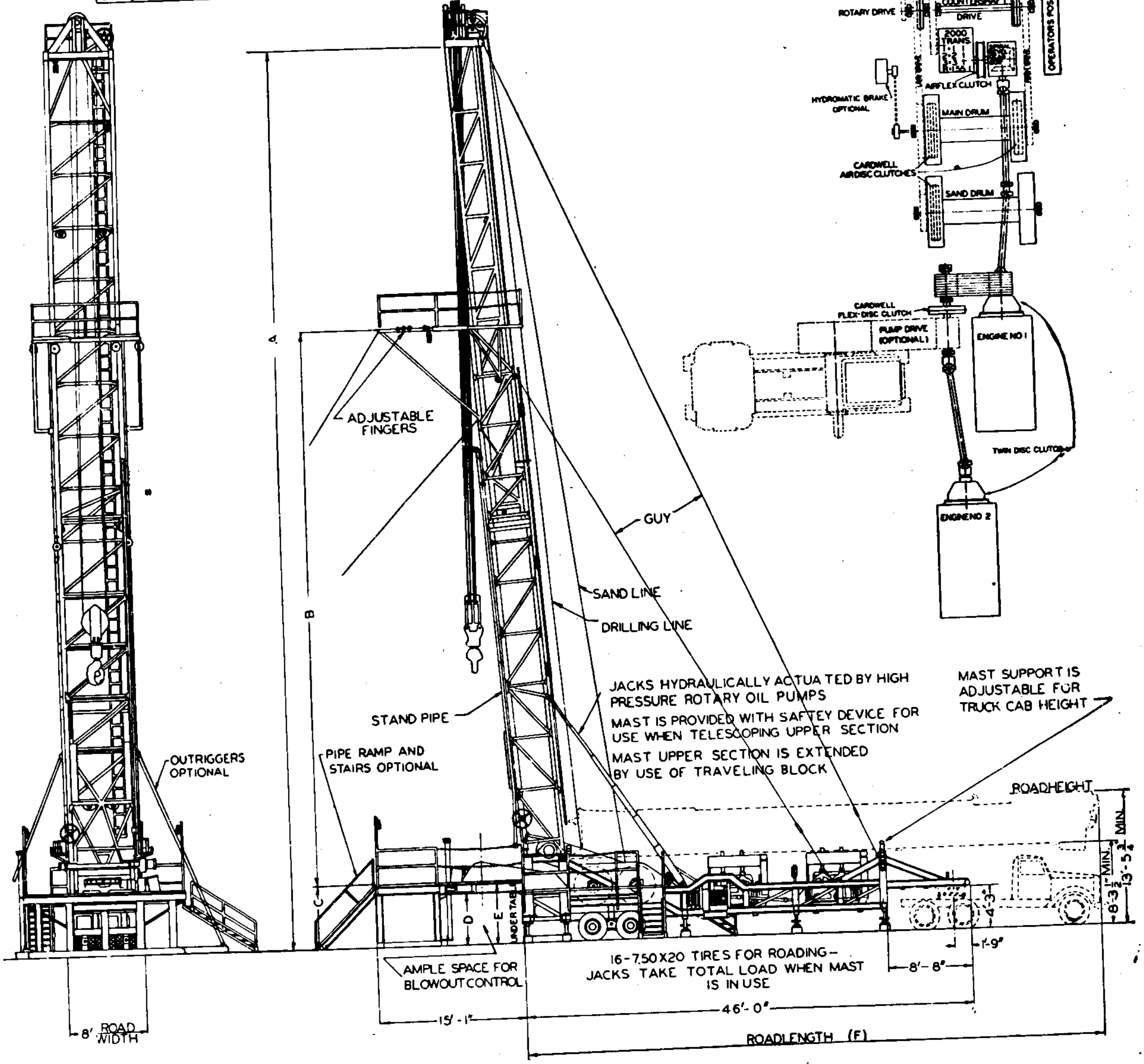


Fig. 5

AN ILLUSTRATION OF A TYPICAL DRILL STRING TOOL JOINT

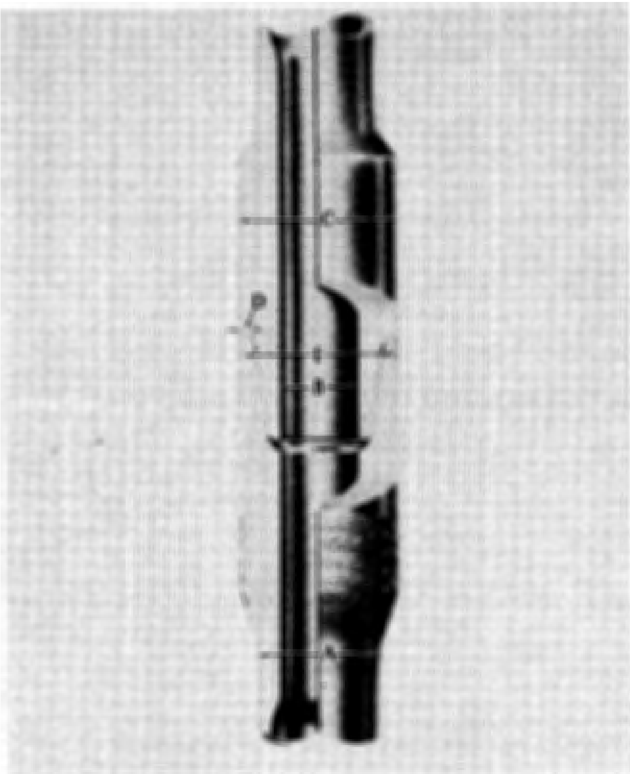


Fig. 6

A PHOTOGRAPH OF A TYPICAL ROLLER CUTTER DRILLING BIT

Fig. 7

AN ILLUSTRATION OF A TYPICAL DRAG TYPE DRILLING BIT

Fig. 8

A SCHEMATIC DRAWING OF THE FLUID FLOW THROUGH A ROLLER CUTTER DRILLING BIT

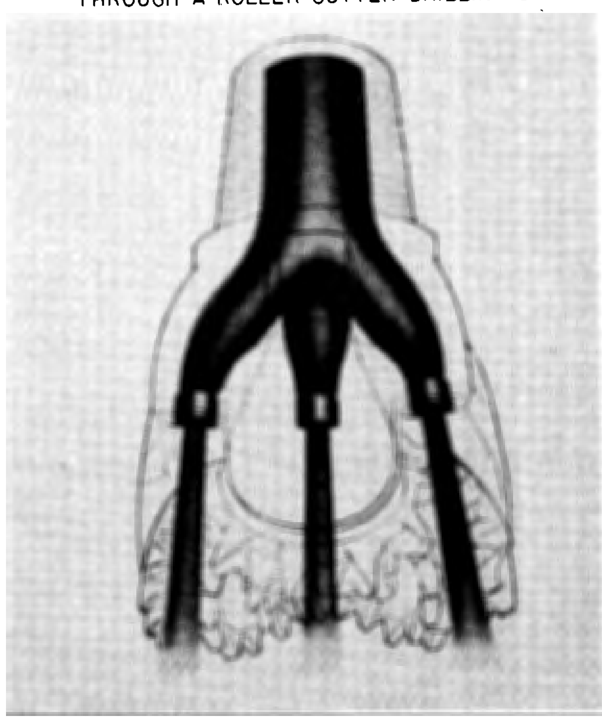
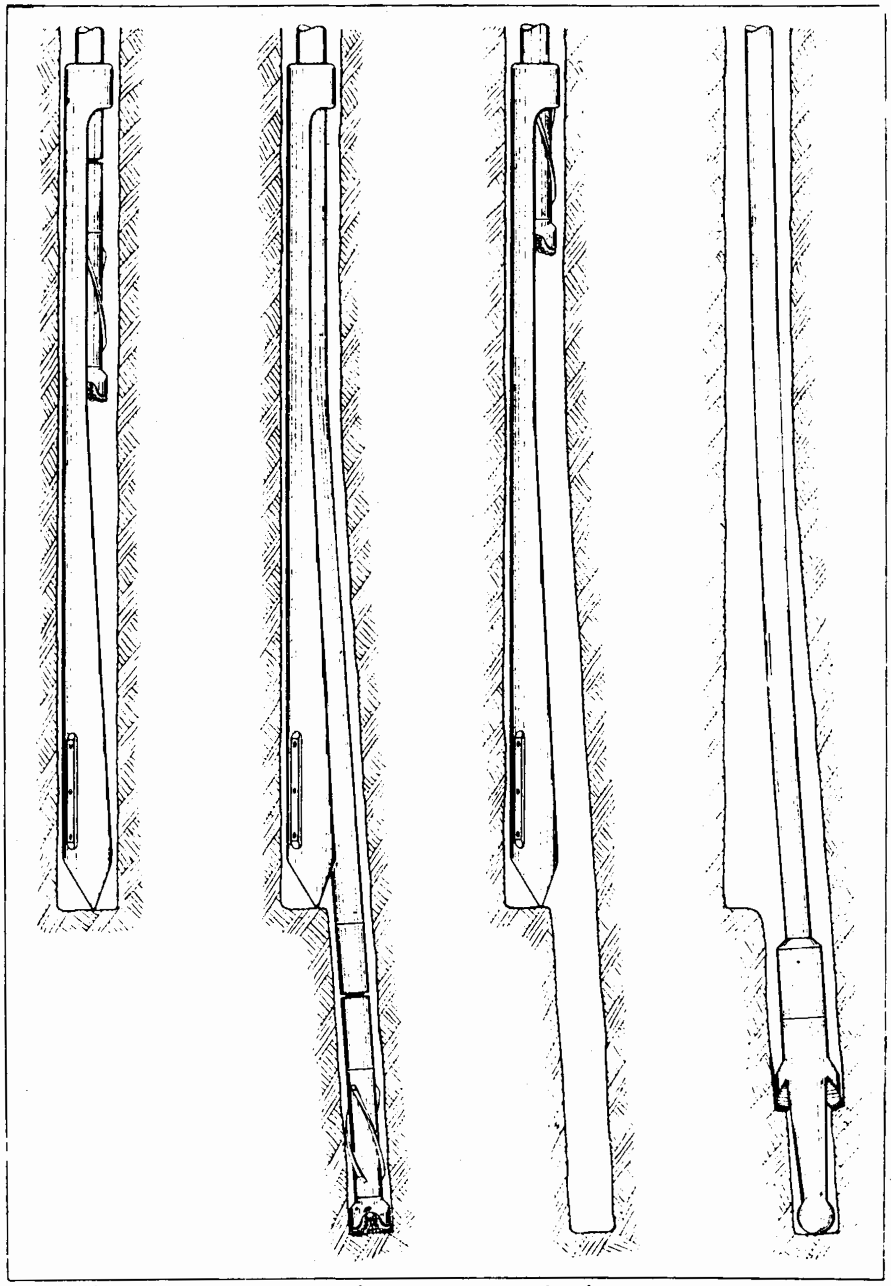


Fig. 9

AN ILLUSTRATION OF THE USE OF A WHIPSTOCK



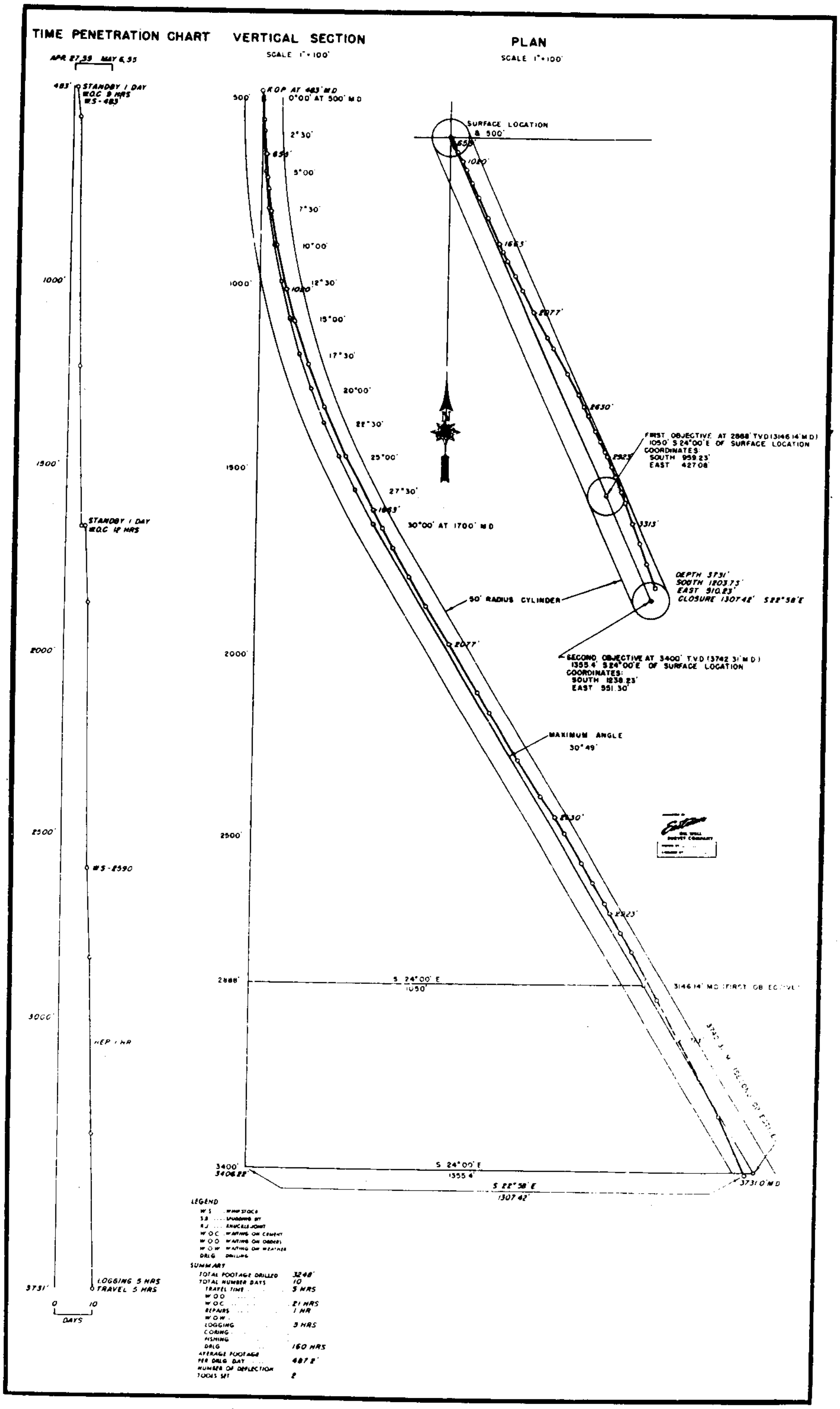
(1) (2) (3) (4)

OPERATION OF WHIPSTOCK

1 — On bottom in oriented position before pin is sheared. 2 — Drilling assembly in rathole. 3 — Whipstock in pick up position. 4 — Reaming rat hole to full gauge with Eastco hole opener.

Fig. 10

A TYPICAL DIRECTIONAL—DRILLING WELL COMPLETION REPORT



Typical directional-drilling completion report.

Fig. 11

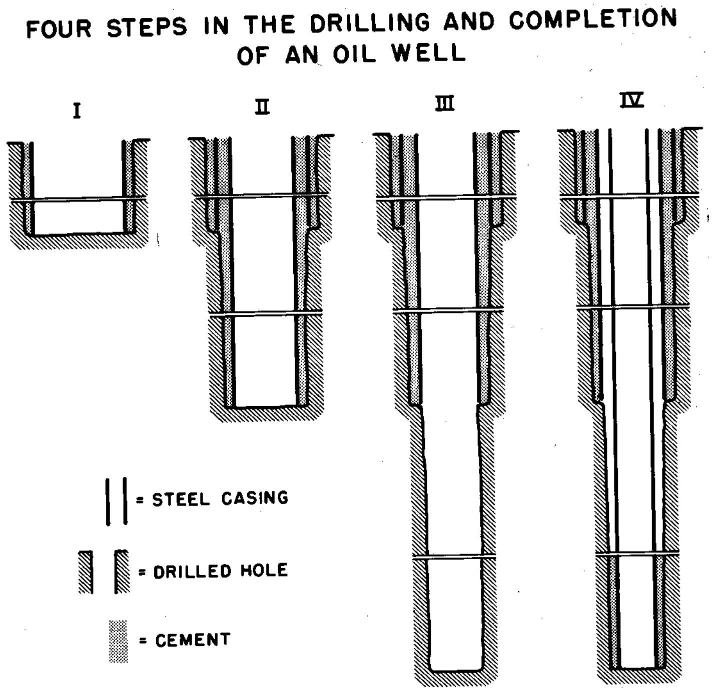

FOUR STEPS IN THE DRILLING AND COMPLETION OF AN OIL WELL
I
II
III
IV
= STEEL CASING
= DRILLED HOLE
= CEMENT

Fig. 12

Fig. 13

RECENT DEVELOPMENTS FOR DRILLING LARGE DIAMETER HOLES

John C. Haspert, Hughes Tool Company
Jack McKinney, McKinney Drilling Company

INTRODUCTION

I shall discuss some of the recent developments in drilling large vertical and horizontal holes, covering the use of oil well drilling methods in particular.

The rolling cutter for oil well drilling made its first successful test at Goose Creek, Texas, in 1909--the first practical application of this concept in fracturing rock. Rock was made to fail by pressure applied against it. Rotary drilling, formerly limited to soft formations using scraper-type tools, was extended through this principle to hard rock. The principle has since been developed into an exacting science. Extensive research facilities are available to work out solutions to new problems, and to apply new knowledge to special applications in drilling.

With rolling cutter bits and the development of other equipment, supplies, and techniques, hole depths have increased through the years. One hole has recently been completed to more than 25,000 ft in depth.

The depth at which drilling is done affects costs and performance, due largely to time loss to replace dull bits and the effects of hydrostatic loads on the rock. Hydrostatic loads prevent some rocks from yielding as easily as they do when drilled at or near the surface (this is important only at depths below 4,000 ft). The work of Eenick, Cunningham, and others in this field has recently been described in AIMME papers and in the oil field trade journals. (See Bibliography, item 1.)

Under good conditions, bit life can be in excess of 100 hours. In extremely hard rock, like some of the taconites in northern Minnesota, the bit life, through extensive research, has been extended from less than 100 ft in 1953 to 800 ft and more in 1958, at drilling rates up to 17 ft per hour.

In dry formations, air might be used more efficiently than water for circulation. It has proven to aid both cutter life and penetration rates.

Development work on the rolling cutter has led to its wide acceptance as a drilling method for all kinds of rock (Bibliography, item 2).

HOW ROLLING CUTTERS WORK

I would like to isolate for a few minutes the tooth on a rolling cutter and its action on a rock formation. Figure 1 shows a tooth applied to a rock with sufficient force to make the rock yield. The amount of force applied generally determines the volume of chips made. The volume increases at a greater ratio than the ratio of any increase in applied thrust. One can readily see that thrust is important to the efficiency of a drilling operation. Drilling performance also varies with every type of rock. Proper technique and cutter selection are necessary to drill each type efficiently.

Figure 2 shows a rock pattern made by a soft-formation rock bit on soft limestone. Notice the tooth impressions. Large size chips were made by a minimum number of fairly large slender teeth contacting the rock. Drilling rates in soft rock with oil field bits having the proper tooth structure often exceed 100 ft per hour.

Figure 3 shows a pattern made by a medium-hard formation rock bit. The rock is marble, which is firmer than the limestone in Fig. 2. The chips were smaller. Cutters to drill this type of formation generally have more and stouter teeth than those for soft formations and perform best when used with heavier drilling weights.

Figure 4 shows a pattern made by a hard formation rock bit in quartzite, a high-strength abrasive rock. The chips were small. Cutters to drill hard abrasive formations have many stout teeth and require the most thrust for efficient drilling.

Figure 5 shows a test which measures wear on a tooth and the amount of rock removed.

SHAFT SINKING

Since 1953 there has been an accelerated amount of development work on rolling cutters for holes larger than those generally drilled by the oil industry. Among these efforts were cutters developed by Hughes Tool Company for core drilling 6-ft diameter mine shafts. A core drilling machine to use these cutters was developed by the Coal State Construction Company, now the Zeni-McKinney-Williams Corporation of Morgantown, West Virginia (Bibliography, items 3 and 4).

Figures 6 and 7 show the original Zeni core drill which to date has sunk fourteen 6-ft diameter shafts to depths of 300 to 600 ft.

Figure 8 shows drilling data from one of the earlier shafts made by Zeni.

Figure 9 shows a recent development by the Hugh B. Williams Manufacturing Company for the Zeni-McKinney-Williams Corporation. This is a "full hole" drilling unit. It is presently undergoing its initial test, drilling two 75-in. diameter holes 500 ft deep into a coal mine near Beckley, West Virginia.

Figure 10 shows schematically the drilling vessel which hydraulically anchors to the wall of the hole and applies 400,000 lbs of thrust to the cutters. It weighs 20 tons. The anchor shoes and hydraulic thrust jacks are controlled from the surface and allow a 5-ft stroke. Drilling rates have been in excess of 5 ft per hour in medium-hard sandstones and limestones and 3 ft per hour in hard quartzites. We do not know what the limit for this type of operation might be.

Figure 11 is a schematic view of the drilling operation. A 12-1/4 in. diameter pilot hole is drilled down into an existing mine entry. Cuttings fall through this pilot hole into the mine for later removal.

Figure 12 shows the surface rig used to drill the pilot hole and to place the vessel into the hole. At first the rotary table was used to drive the vessel through a drill stem. Recently motors to provide rotation have been placed directly on top of the vessel. With this change the vessel can ultimately be lowered or taken from the hole with a wire rope rather than by handling a long drill stem.

Figures 13, 14 and 15 show the new Zeni-McKenney-Williams portable drilling assembly being readied for moving. The entire assembly can be set up for drilling or torn down for moving in 4 to 6 hours.

Figure 16 shows a Calyx drilling system. The Calyx basket above the bit collects the cuttings. The basket and bit are withdrawn for removal of the cuttings.

Figure 17 is a view of a Calyx system planned for drilling a large hole to 800 ft deep in granite. Instead of a short basket, a 25 ft or longer basket may be used. The cuttings collected in the basket will slightly increase the drilling thrust.

Figure 18 shows a reverse circulation set-up. Mud is allowed to flow down the hole annulus and is pumped out through the drill stem either by pump, air lift, or eductor. This system can be used in weak or water-bearing formations. The rig shown has a rotary table high enough for large bits and other equipment to be moved in and out of the hole easily.

Figure 19 shows a reverse circulation drilling rig in Europe which is producing 1500 ft deep holes 25 ft in diameter through successive stages of reaming.

Figure 20 shows a machine now in use in Europe. A small 8-in. pilot hole is drilled from a mine entry upward to the surface. A drill stem through the pilot hole is then used to pull a 32-in. bit down in a reaming operation. Drilling from below provides an effective method for removing cuttings, which drop and are deflected into mine cars. A pull-down operation of this type simplifies equipment that can develop sufficient thrust. This can be done in reverse as shown in Fig. 21.

Figure 22 shows a boat pier construction system--an unusual experiment presently being developed with the British. Rolling cutters normally used for core drilling are mounted on the bottom of large-diameter casing. The casing is then used as a drill stem. After the desired depth is reached, the casing with the cutters is cemented in place.

<u>TUNNEL DRIVING MACHINES</u>

No completely successful hard-rock tunneling machine has been built for large diameters. We feel, however, that the rolling cutter application will be as helpful to the tunneling industry as it has been to the oil well drilling industry.

Tunnel drivers have been successful in soft formations but all efforts so far in hard rock have been generally underpowered, lacking in thrust capacity, and have had cutter designs which would not stand up.

Figure 23 shows a 42-in. model of a hard-rock tunnel driver about to be field tested. From this, machine design criteria will be developed for an 8- to 12-ft machine. From information already developed it is believed tunnels in these diameters in any rock will be drilled 2 to 3 times as fast as by conventional means and at equal or lower costs. Tunnels drilled with rolling cutters would have the following advantages:

1. Higher rate of advance
2. Less disturbance to wall (see Fig. 24)
3. Less tunnel maintenance
4. Less overbreak
5. Safety
6. Less labor and overhead

Some disadvantages would be:

1. High equipment cost
2. Difficulty in making turns
3. Limitations on shape
4. Bulky equipment difficult to move

Figure 25 shows a burn hole operation. The center is drilled with a conventional-type oil field bit. A jumbo drills the small surrounding powder holes. Shooting with this arrangement generally provides better advance per round and better fragmentation.

Consolidated Edison Co. of New York has developed the machine shown in Fig. 26 for drilling up to 42-in. diameter horizontal holes into hard rock. The machine is being used by Boring, Inc., another New York company, to drill horizontal holes under New York City and other locations in the state. It is capable of developing up to 200,000 lb thrust.

Research and development work is progressing at a good rate, and we plan to continue to further demonstrate the practical applications of rolling cutters to drilling large holes. We have in the planning stage a number of ideas that may be practical for hole requirements larger than those discussed in this presentation. The proposed hole diameter, the type of rock structure, and the formation conditions would greatly influence the design and construction of the drilling equipment.

BIBLIOGRAPHY

1. Cunningham, R. A., and Eenink, J. C., "Laboratory Study of Effect of Overburden, Formation and Mud Column Pressure on Drilling Rate of Permeable Formations," Journal of Petroleum Technology, January 1959.

2. Hughes Tool Company, Rotary Drilling Bits, 1959.

3. Zeni, Victor, and Williamson, T. N., "Sinking Large Diameter Mine Shafts by Rotary Drilling," Mining Engineering, April 1957.

4. Morlan, E. A., "Oil Field Bits in the Mining Industry," The Mines Magazine, November 1956.

FIGURES

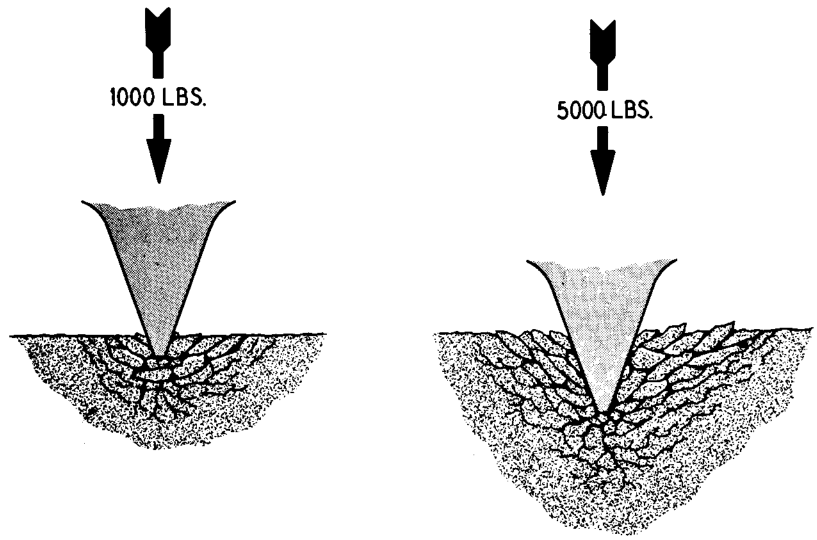


Fig. 1

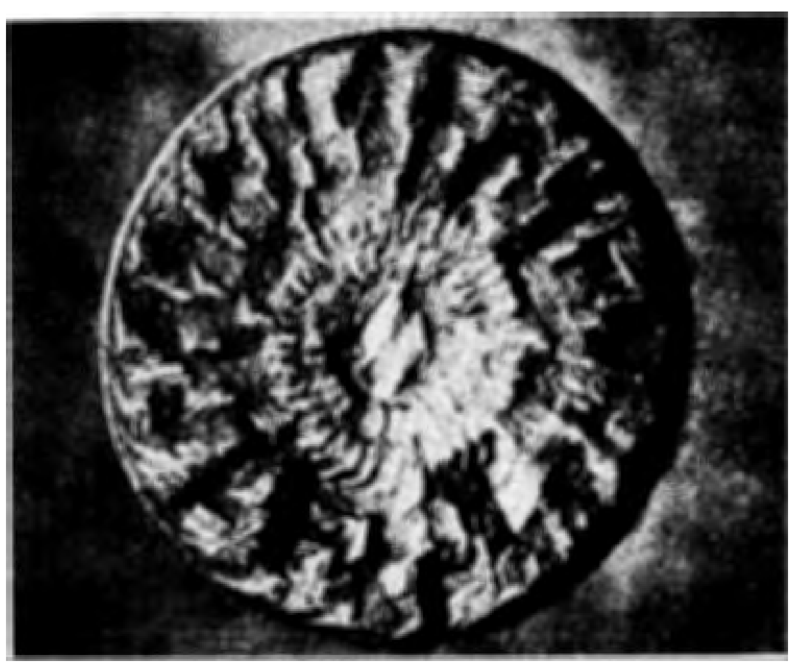


Fig. 2

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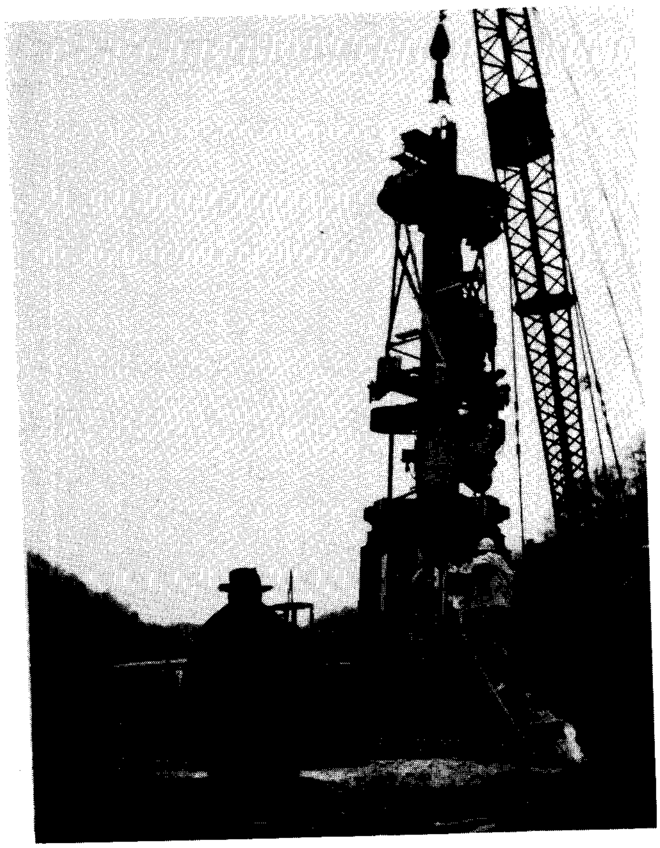

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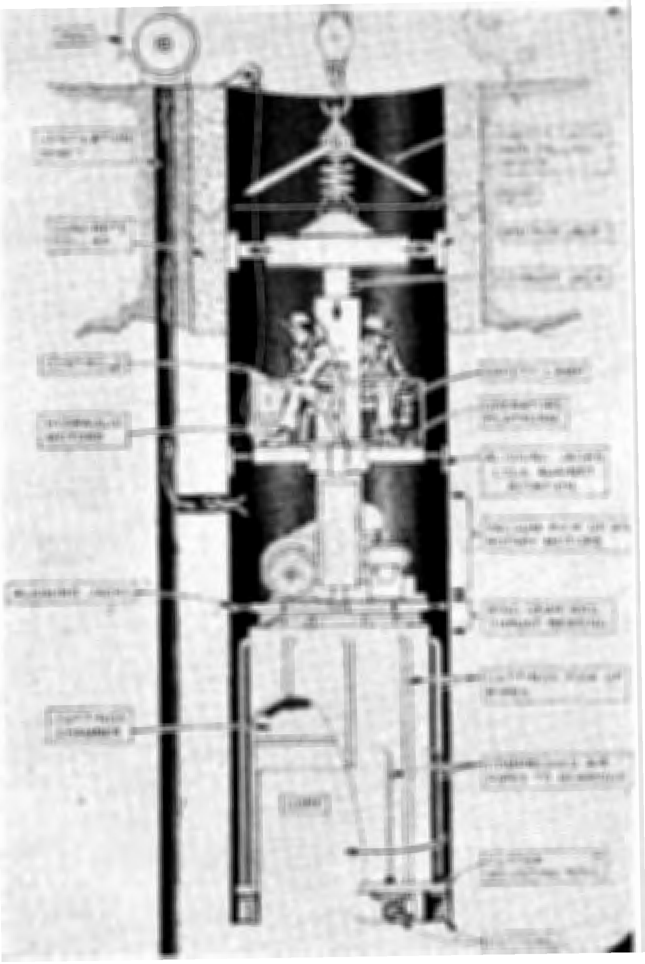


Fig. 7

PROGESS DATA-SHAFT NO. 6	
DIAMETER	75
DEPTH SHAFT FEET	464.8
DEPTH DRILLED FEET	452.5
TOTAL ELAPSED DAYS (SINGLE SHIFT) INCLUDING HOLIDAYS	79
TOTAL DRILLING DAYS	55
FEET PER HOUR INCLUDING DOWN TIME	0.93
FEET PER HOUR DRILLING TIME ONLY	3.14
FEET PER 8.8 HOUR SHIFT	8.23
FEET PER SET OF CUTTERS	51.9
FEET DRILLED PER TRIP	3.62
TRIPS PER SHIFT	2.27

(DATA COMPILED BY ALBINO ZENI FOR JOB AT BUNKER MINE NO. 2, TROTTER COAL CO. AT CORE, WEST VIRGINIA,- SHAFT COMPLETED 8-15-56.)

Fig. 8

Fig. 9

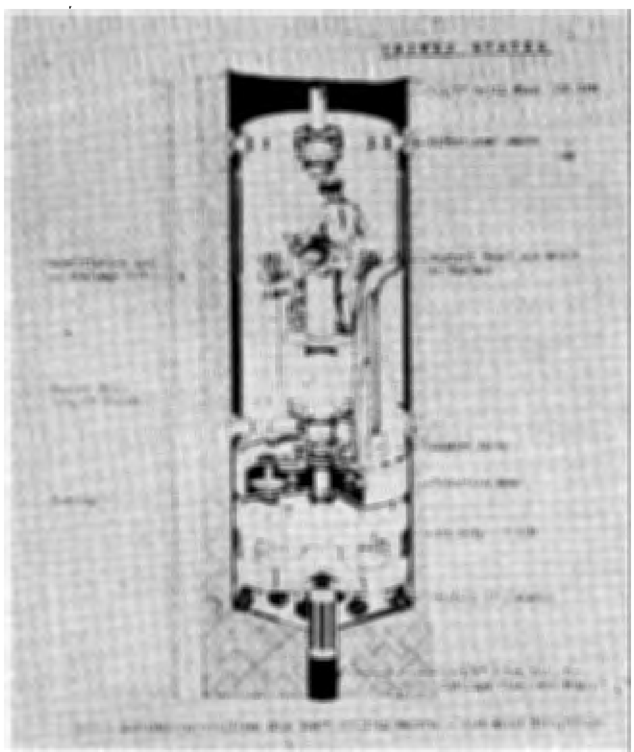


Fig. 10

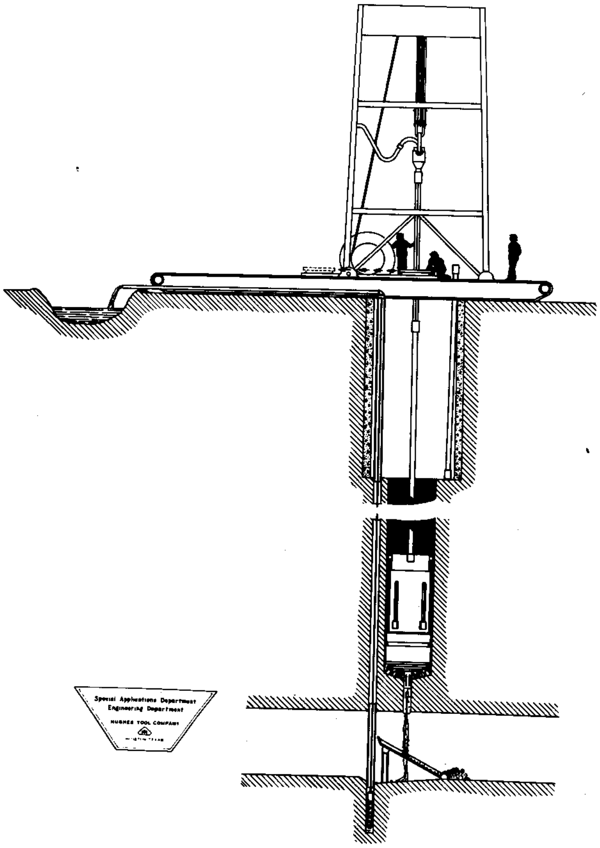
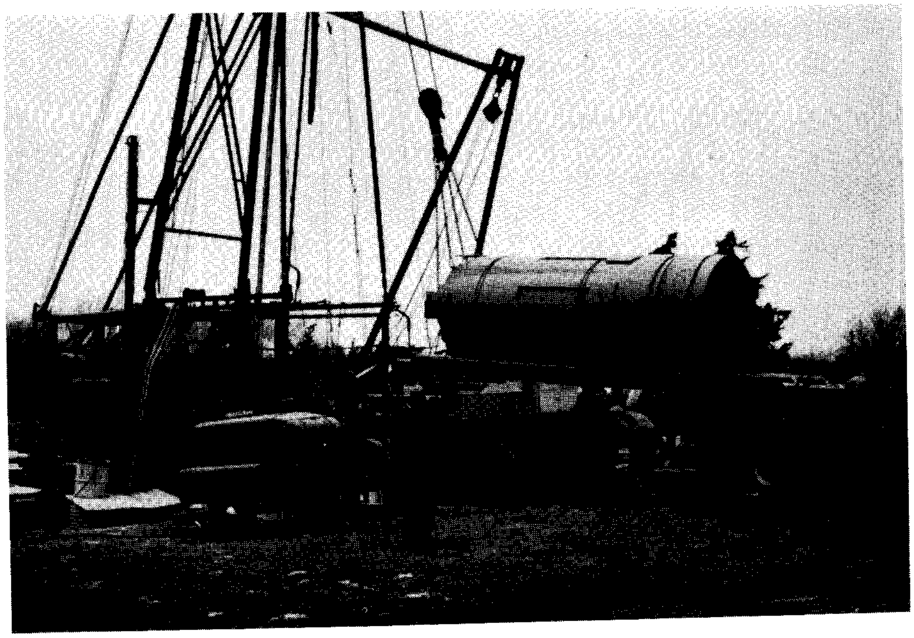

Special Applications Department
Engineering Department
HUGHES TOOL COMPANY

Fig. 11

Fig. 12



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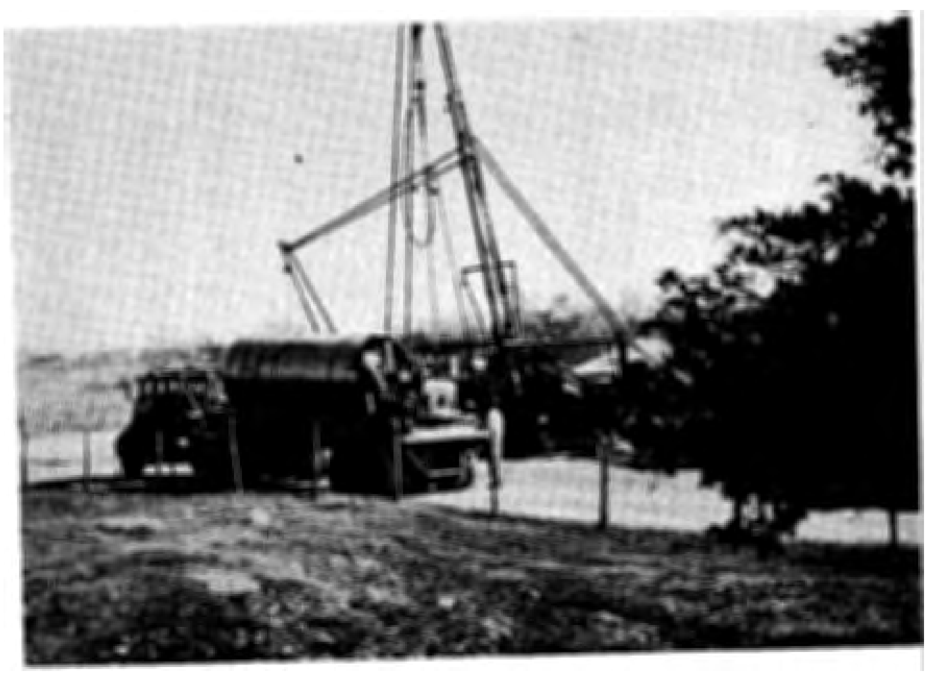
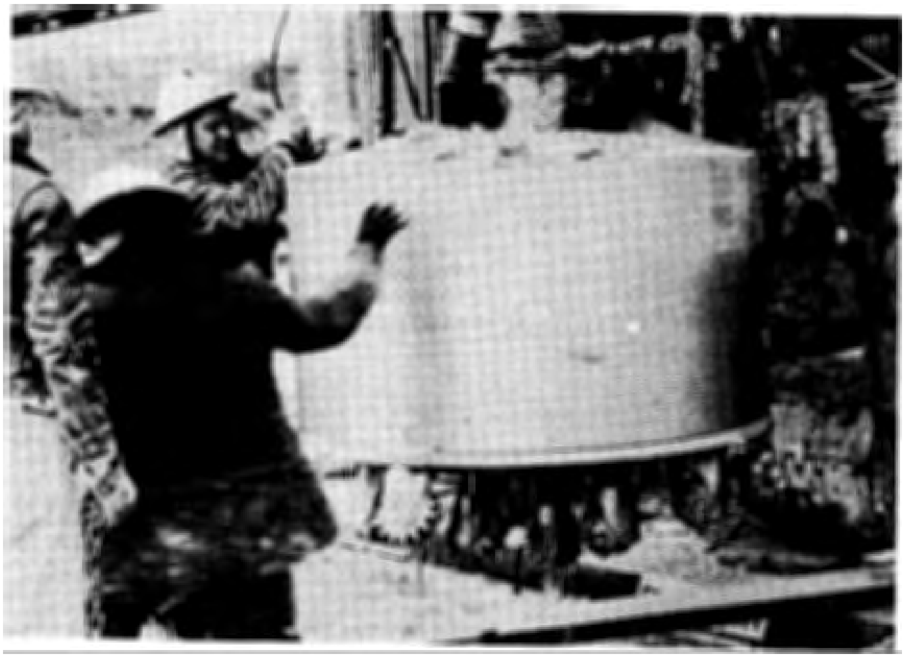


Fig. 14

Fig. 15



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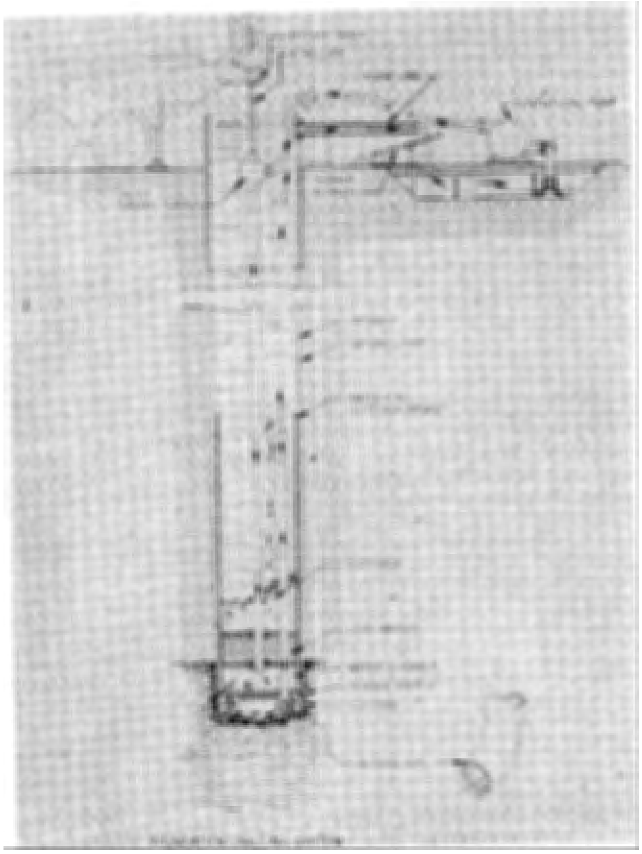

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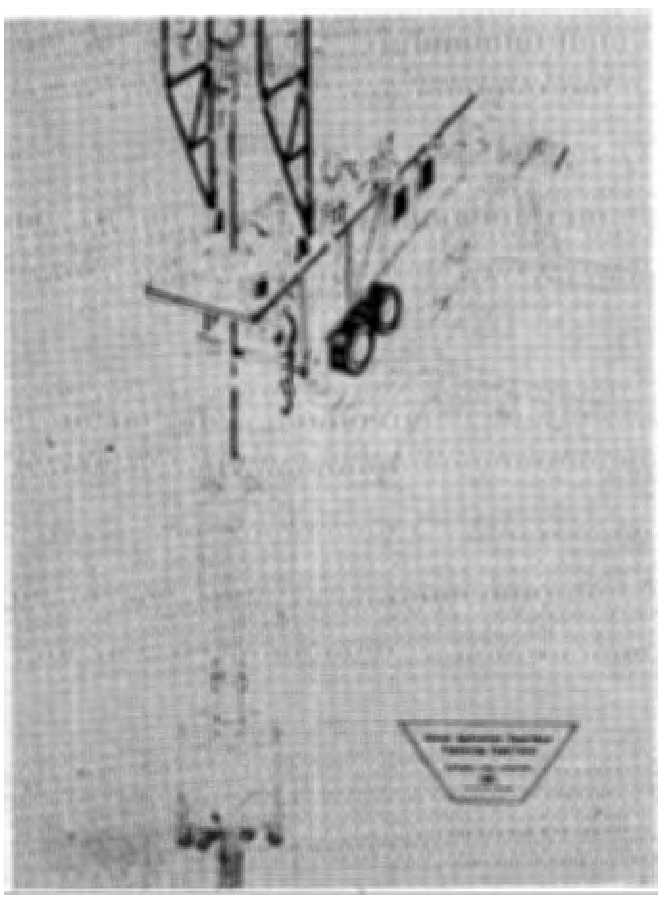
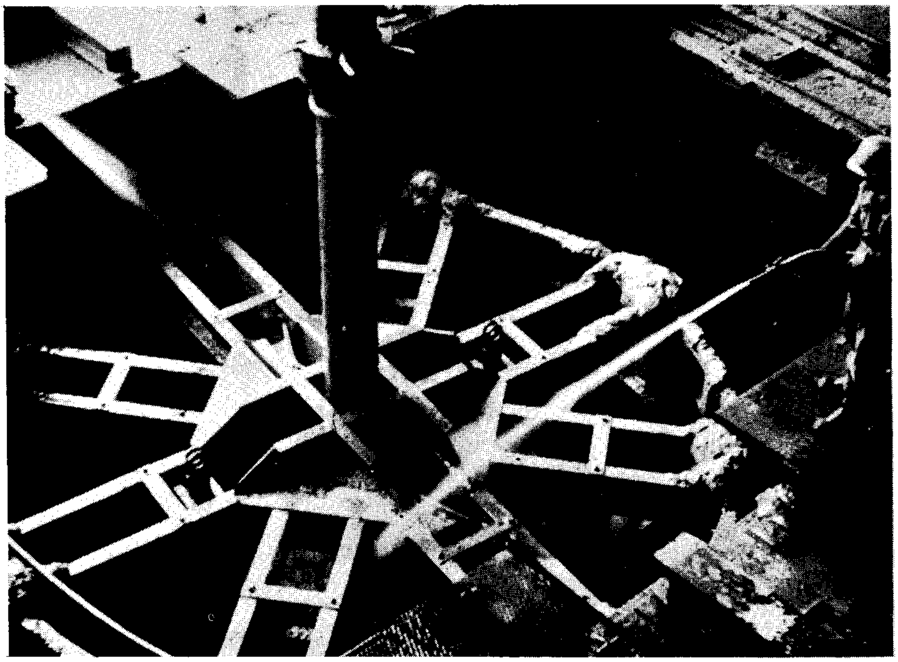


Fig. 18



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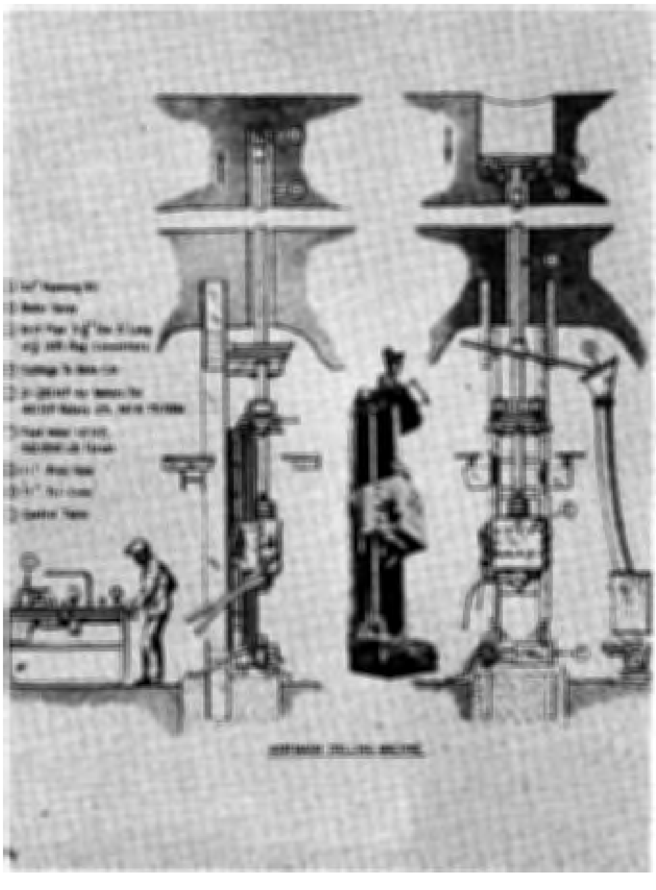

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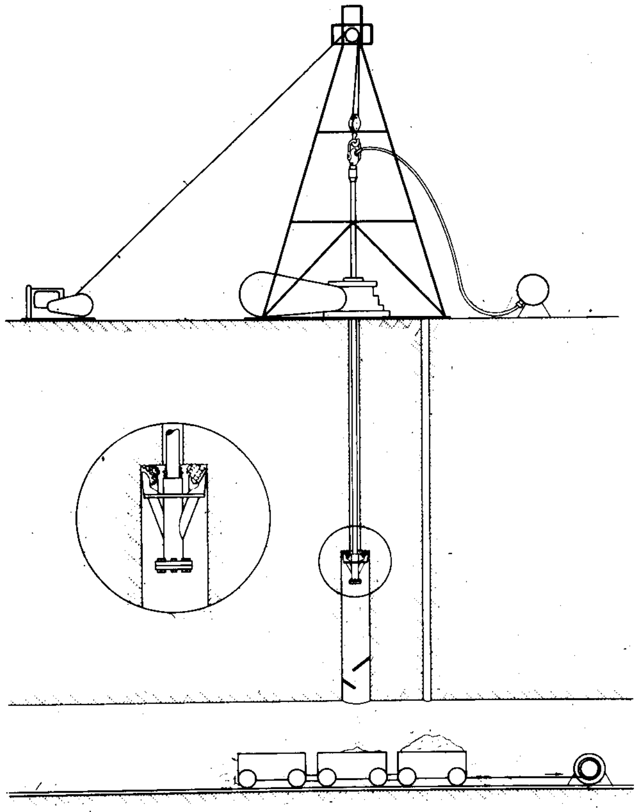

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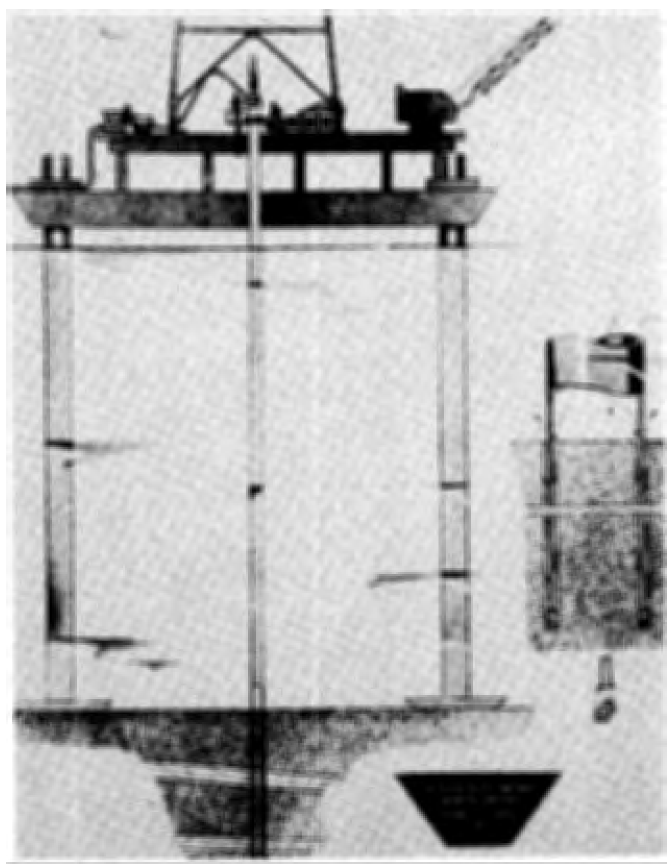

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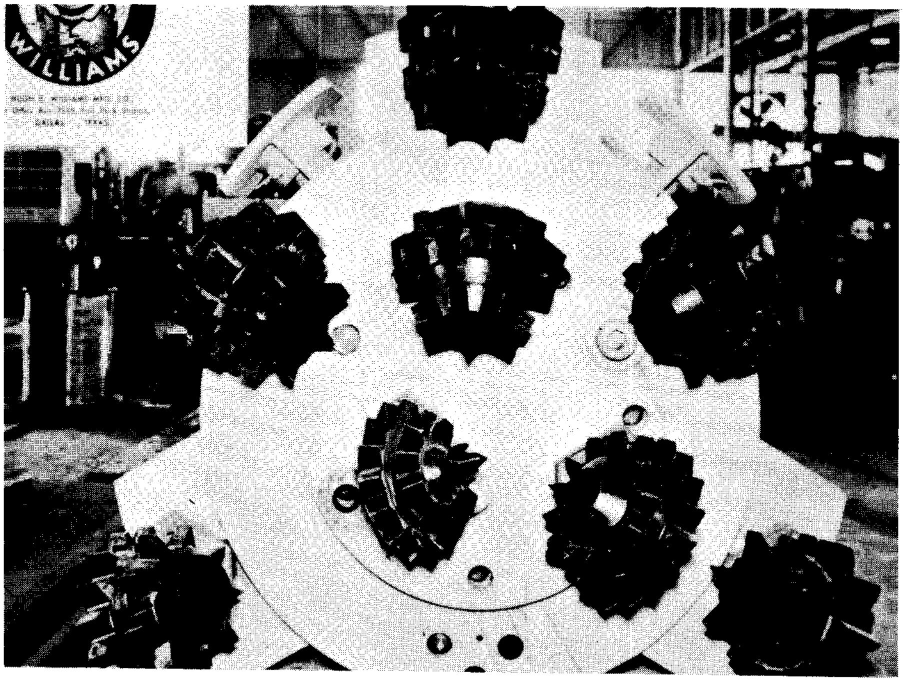

Fig. 23

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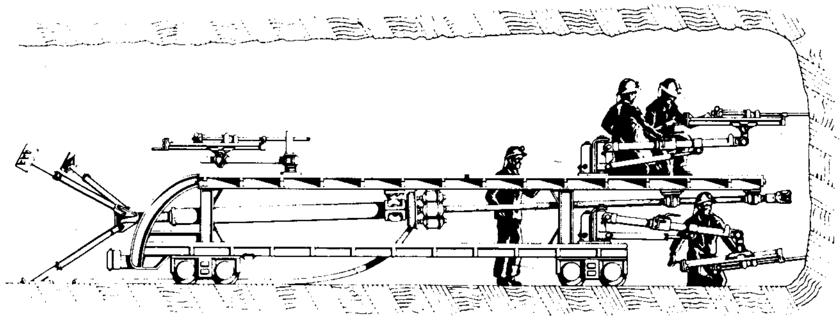

Fig. 25

Fig 26

LARGE TUNNEL MACHINES

Richard J. Robbins
James S. Robbins and Associates

Until the introduction of black powder from China, most tunnels were driven by continuous operation, that is, hammers, picks and shovels. Another ancient method of rock breaking was sudden quenching of a heated rock surface. This marked the beginning of a trend to a cyclic mining or tunneling operation.

Today, normal tunneling follows an operating cycle made up of the following steps:

1. Set up and drill
2. Load and blast
3. Ventilate
4. Bar down loose material and muck out
5. Timber and lay tracks

The cyclic mining method has been mechanized and improved to the point that, today, it is possible for a highly proficient crew to complete 3 cycles in one shift and, under good conditions, a tunnel crew can complete 50 to 60 feet in 3 shifts.

In recent years there has been a revival of "thermal" methods of rock breaking in the form of jet piercing and sonic and ultra-sonic pulsing of atomically disassociated gases, such as fluorine.

Pacing this revival of ancient tunneling methods, James S. Robbins & Associates have gone back even further than the thermal breakdown of rock, to mechanical methods. We have developed continuous mining and tunneling machines that have made substantial improvements over the highly developed cyclic methods commonly used.

Many of the advantages of the continuous mechanical method are obvious. I shall describe some of the more important ones.

1. Safety. Frequently a bored tunnel requires no roof or wall support. Even in very bad ground a minimum of roof bolting and timbering is required.

2. Save on outbreak. A tunnel can be bored to a size within an inch or less of actual required dimensions, resulting in large savings on grouting or concreting. It is also possible to remain precisely on line and grade. Common outbreak in blasted tunnels is from 9 to 10 in. This will increase the concreting cost by as much as 100 per cent over that on a smooth bored tunnel.

3. Lower labor cost. A much smaller crew is required to bore a tunnel than to drill and blast one.

4. Rapid tunnel advance. Even our first applications of the continuous boring technique have resulted in tunnel advances far exceeding those ever attained by the use of blasting.

5. Improved haulage. Because a bored tunnel can approach a continuous operation, a more efficient method of haulage can be acquired.

6. No property damage liability. Continuous tunnel boring eliminates the blasting claims in industrial or residential areas.

Some present disadvantages of the system are:

1. Possible limitations as to the type of rock that can be bored. Hard and flinty, or tough and fibrous formations such as tachanite, some basalts, granites and other igneous rocks might remain beyond the capability of our Mechanical Moles. These rocks comprise a very small percentage of the earth's crust.

2. Higher initial capital expenditure. Generally, this machine must be amortized on the single project for which it was built. Until

we can supply some machines on a rental basis, their use on very short tunnels will not be economically feasible.

3. Working on a curved tunnel bottom. This is a disagreeable problem in a small tunnel. We are now developing a machine which will modify the circular cross section to one of an oval or semi-horseshoe shape, where those shapes might be of advantage.

Many of the ideas that led to the development of the Robbins Mechanical Mole are not new, but the fact that they have been employed with economic success is new. I shall review a brief history of the development of the Robbins tunneling machines.

A continuous mining method has intrigued man since his first efforts at underground excavation. An outstanding effort took place in England in the 1870's. A machine, the McKinlay entry driver, was built and actually bored a mile of tunnel in chalk. This was the proposed English Channel Tunnel. Had such materials as borium and tungsten carbide been available at the time, the venture probably would have been an economic success.

While Mr. Robbins was working as a consultant for a group of coal mine operators in southern Illinois, he came across a McKinlay Miner which had been abandoned as an impractical idea. This machine seemed to have some basic design advantages over the Joy and Jeffries machines being developed at that time. Mr. Robbins' ideas for developing a continuous miner of the McKinlay-type proved highly successful. The Marietta Mfg. Co. built several of these machines, which became known as the Marietta Miner.

In 1952 the patents for this machine were sold to the Goodman

Manufacturing Company of Chicago. Mr. Robbins worked for the Goodman Company as a consultant, redesigning and developing the Goodman Miner, which in some instances, more than doubled the daily production of any other mining machine or method known. It soon became possible for a Goodman machine, with one operator and one helper, to average over 2000 tons of coal per day on a two-shift operation. Thus, one machine now produces as much as a large size coal mine in the past.

The success of this type of machine in coal led to the attempt to use it for slope-driving in rock. Two 2800-ft long haulage ways were to be driven at a down-grade angle of 16½ degrees. These tunnels were successfully driven through sediments ordinarily requiring drilling and blasting. The disc cutter was developed in the process of driving these tunnels.

It was found that rock, being sufficiently harder and less brittle than coal, required a disc cutter to be rolled on the rock surface between the kerfs cut by the fixed cutters. This disc cutter action was similar to that of the disc used in a glass cutter. The disc creates point pressure at the rolling contact of the disc on the rock, spalling the rock outward, toward the kerfs. The use of the disc cutters reduced the power requirements, increased the penetration rate and produced good chip size.

Another of these machines was used to drive a tunnel in shale under the Arkansas River. In the early stages, only the very softest formations were bored with economic success, and the economy of the formation to be bored would not stand up well enough for blasting.

Development of the Robbins Mechanical Mole

Early in 1954, the Mittry Construction Company of Los Angeles secured a contract for driving the Stage 1, 25 ft 9 in. diameter tunnels for the Corps of Engineers Oahe Dam Project in Pierre, South Dakota. Mr. F. K. Mittry decided that a mechanical mining method could be used to advantage in the Pierre shale and Crow Creek marl formation. James S. Robbins & Associates were given the job of developing the design and building a machine. The engineering and design of the 125-ton machine cost approximately $85,000.

After an extensive "debugging" process, the Mittry machine, commonly referred to as the "Mittry Mole," began setting tunneling records and cutting costs of excavating to an unprecedented degree.

The Oahe Dam job involved many complications, due to the especially bad roof conditions and stringent safety regulations of the Corps of Engineers. The specifications required that rolled H-section ring beams be installed to support the walls and roof of the tunnel every 4 ft. This meant that we had to mechanize the installation of the ring beams to keep pace with the advance of the tunnel boring machine. Also, part of the tunnel was to be driven on a gradual curve and it was necessary to keep absolutely on sites. The specifications would permit the rings to be not more than 1 in. out of alignment, sideways or vertically. It was required that the tunnel remain at all times at 95 per cent humidity to prevent deterioration of the rock wall, and a 2½ ft thick concrete lining had to be placed over the original exposure within 30 days. It was necessary to stop the tunneling operation periodically and follow up with the concrete lining.

The Mittry crew accomplished wonders in coping with these handicaps, and their best 8-hour shift produced 61 ft of bored and re-inforced tunnel. In an engineering time study the operation was observed for six consecutive shifts. The average progress per shift was 40.5 ft. However, the total operating time was only 51 per cent of the possible operating time. The 49 per cent down time was due primarily to resetting the propelling jacks, inability to supply the material to the machine, cave-ins of the roof and face ahead of the ring beams, and an overloaded conveyor system.

Power consumption averaged about 200 HP or 45 per cent of capacity.

The number of fixed cutter bits consumed was 362, or about 1.5 bits per foot of tunnel. No disc cutters were consumed. The operation required seven men, including a superintendent. In addition, 15 men were required to place the ring beams, weld them and gunnite. The entire crew inside the tunnel, and a supporting crew outside the tunnel for service and muck haulage, was 29 men. This tunnel crew drove 635 ft in a six-day week. With a more adequate haulage system and better roof support, this rate of advance might have been doubled. The machine actually bored from 8.5 to 12 ft per hour while it was operating.

Thus far, James S. Robbins & Associates have built seven tunnel boring machines. As our confidence grew in our tunneling method, we tackled jobs involving harder rock. With our early machines, we attempted to bore some rock which did not prove economically feasible until our later machines were developed.

The fifth machine built was designed to bore sewers in a hard Chicago limestone of 18,000 - 24,000 psi compressive strength. With this machine, it became quite evident that the fixed cutters were not suitable for cutting the harder formations. While the tungsten carbide inserts are hard enough to cut the rock, the drag-type tool bits were subject to excessive shock loading, and it was impossible to keep the carbide inserts in the tools.

Our sixth machine, 10 ft 9 in. diameter bore, was built for the Foundation Company of Ontario and is now boring the sewers under the city of Toronto. This machine is cutting shale interbedded with well-cemented sandstone and hard crystalline limestone.

The Toronto machine marked our second major breakthrough in cutter tool design. Previously, the replacement of fixed cutters was a major cause for down time and the highest replacement part cost. The disc cutters were so arranged as to eliminate the drag-type fixed cutters except for a small number in the central portion of the cutterhead. Although the machine was supplied with fixed cutters and discs, it was found that when the fixed cutters were removed the machine would bore the rock at the same rate with discs alone.

The Toronto machine, which weighs 65 tons, and cost about $175,000 is advancing at a rate comparable to the larger machines in the soft Pierre shale. This machine is cutting rock which is harder than many of the rocks which early machines could not cut economically. Performance data and actual cutting rates of Model #131 are under a cover of industrial security at this time and cannot be released.

Our seventh machine, designed and built for Morrison-Knudsen-Kiewit-Johnson, was just completed and assembled at the job site last month. It has not been operating long enough to give us more than an indication of its capabilities.

After going through a minor "debugging" process, the machine got under way, boring into very messy rock with threatening roof conditions. The machine has mined at the rate of 12 ft an hour for short periods, but has had to slow up for conveyor and skip hoist problems since the material handling crew is not thoroughly broken in.

This is the third big machine that has been delivered for the Oahe Dam Project, but it is different from the first two machines in many ways. The machine is capable of placing the ring beam supports closer to the tunneling face and it has a front end, sideways and vertical adjustment which keeps the machine on sites and helps make shorter-radius turns.

Unlike the first two Oaha machines, the cutterhead on this Model #351 is a single unit rotating in one direction. This reduced many mechanical problems in design and fabrication. On the smaller machines, the torque reaction for the cutterhead which turned in one direction was taken up by the side wall gripping shoes. On the Model #351, the dead weight of the machine and Jumbo, supported by the Dollies, supplies the reaction. The machine is removing 4 to 5 cubic yards of in-place rock per minute, and when the operating and supporting crews are well-seasoned they will be able to sustain this rate over a long period of time.

Present Improvements and Future Developments

James S. Robbins & Associates is now designing a more versatile machine which will be able to tunnel through almost any type of rock that might be encountered. Our present proven capability includes all of the softer formations and most of the harder sedimentary rocks such as hard limestone, sandstones and mudstone. These rocks comprise about 75 per cent of the earth's crust. About 65 percent of the dry surface of the earth is made up of shales, all of which are well within our capabilities. We believe our present machines are now capable of cutting many of the metamorphic rocks and some of the softer igneous rocks. Improved cutter designs and layout configurations have been drawn up but not tested.

Costs and Performances

The Robbins Mechanical Moles have cost from $150,000 to $500,000, depending on the size and special requirements of the rock and tunnel design. A 15-ft diameter machine might cost between $250,000 and $350,000.

Our machines have progressed at a rate of 10 to 12 ft an hour in the hardest formations we have encountered, and we can see that in softer formations with adequate haulage systems and no problems in roof support, tunneling rates of 20 ft an hour may be obtained. Average rates of progress, regardless of the size of the tunnel, have been between 30 to 50 ft per shift.

Bit consumption costs, which were at one time a major factor, have been virtually eliminated since our switch to an all-disc cutter configuration. An early machine bored four 1800-ft tunnels and required the replacement of only six discs, and these due to seal failure on the bearings and not to excessive wear on the disc edge.

When using a Robbins Mechanical Mole, general excavation costs can be based primarily on the cost of the labor and power, and write-off of the cost of the machine. A machine for a 12-ft diameter tunnel might cost $200,000. For a 5000-ft tunnel, using an average rate of advance of 5 ft per hour, the excavating costs would run about $11.50 per cubic yard or $48.00 per foot. Since the major cost is write-off on the machine, these costs, for a 10,000-ft long tunnel, would drop to about $6.75 per cubic yard or $28.00 per foot of tunnel. This includes the labor cost of a five-man crew, but does not include haulage or roof support labor, or equipment. This bit of "horseback" engineering indicates a boring machine can be used economically, and amortized, even on short tunneling jobs. In addition to savings on excavation costs, a bored tunnel of 12-ft diameter will save an average of one yard of concrete per foot of tunnel, depending on the expected outbreak in a blasted tunnel.

Large-Bore Vertical Shaft-Sinking Machines

If a Robbins Mechanical Mole is upended and supplied with a suitable conveying system, it might become known as a Robbins Continuous Shaft-Sinking Machine. This oversimplifies the design of a shaft-sinking machine, but several designs for both continuous and intermittent shaft-sinking machines have been made.

The possibility of many defense missile launching sites involving underground installations has sparked the development of several new shaft-sinking methods. Recently, James S. Robbins & Associates applied for patents on methods which would allow a mobile unit, including a shaft-sinking machine, to be quickly towed to the site, positioned and begin operation. This machine should be capable of sinking shafts at about the same rate that our tunneling machines bore tunnels, and at about the same cost per foot. A shaft 150 ft deep might require several days to sink. The machine would then be quickly retracted and rolled off to the next shaft site.

We have just prepared cost estimates on using a continuous shaft-sinking device for a contractor bidding on an Air Force Titan installation in Colorado. This one machine would bore tunnels varying in diameter from 10 ft to 45 ft.

Just as airplanes, ships and automobiles have given man mastery of the surface of the earth, tunnel-boring machines and shaft-sinkers will give him access to the subterranean world. It is our aim to provide machines which will supply the ever-increasing demand in mining and construction of underground facilities. We expect to play a major part in mechanizing the excavating industry.

DISCUSSION

MR. WILLIAM W. PLEASANTS (RCA, Moorestown, New Jersey): I think you said you were not able to cut through granite.

MR. ROBBINS: We haven't attempted it yet. Each of these machines that we have built has been built for a specific job. Unfortunately, the contractor has not loaned them back to us for a month or so to test them in any different kind of rock. We can base our ability to cut certain types of rock only on laboratory tests. We believe that as soon as we get a chance to bore some of the harder rock, we will be able to prove our capability in them.

FIGURES

25. Half of cutterhead support.

26. Assembling cutterhead on cutterhead support for testing.

27. Final assembly of cutterhead on machine.

28. Operator's console of Model 351--42 controls.

29. Model 351 completely assembled.

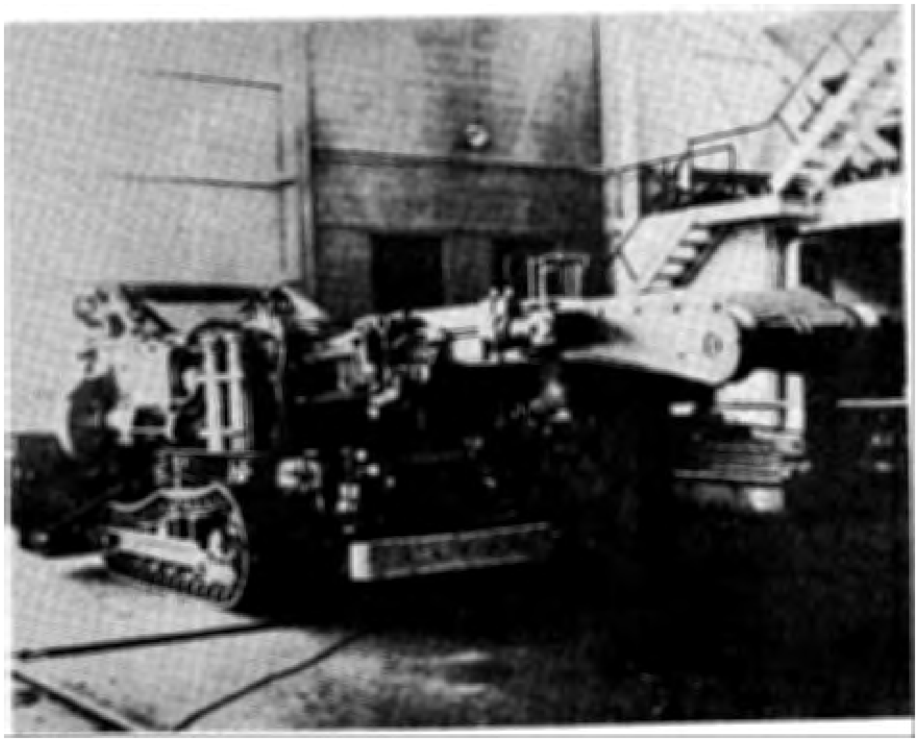

Fig. 1

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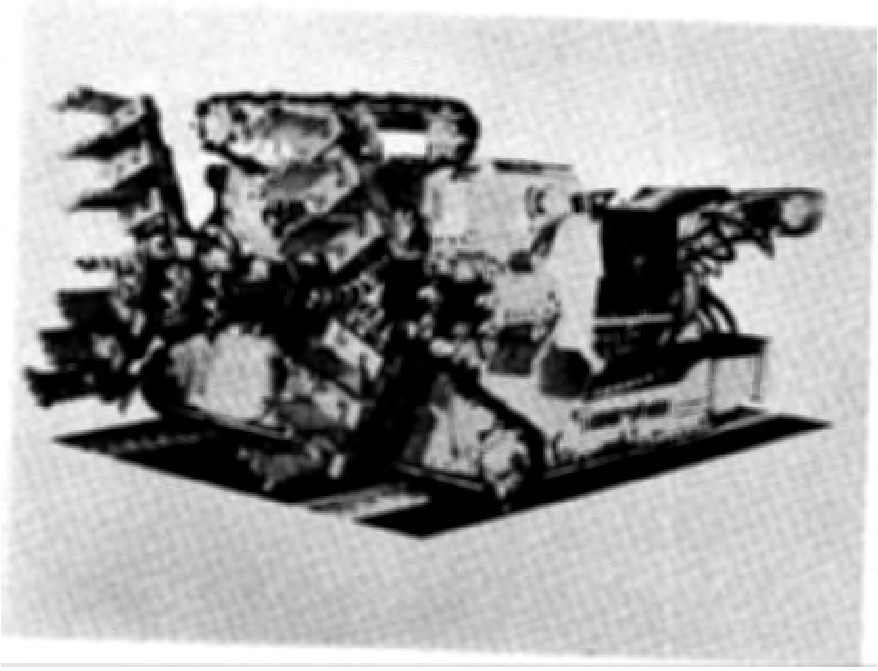

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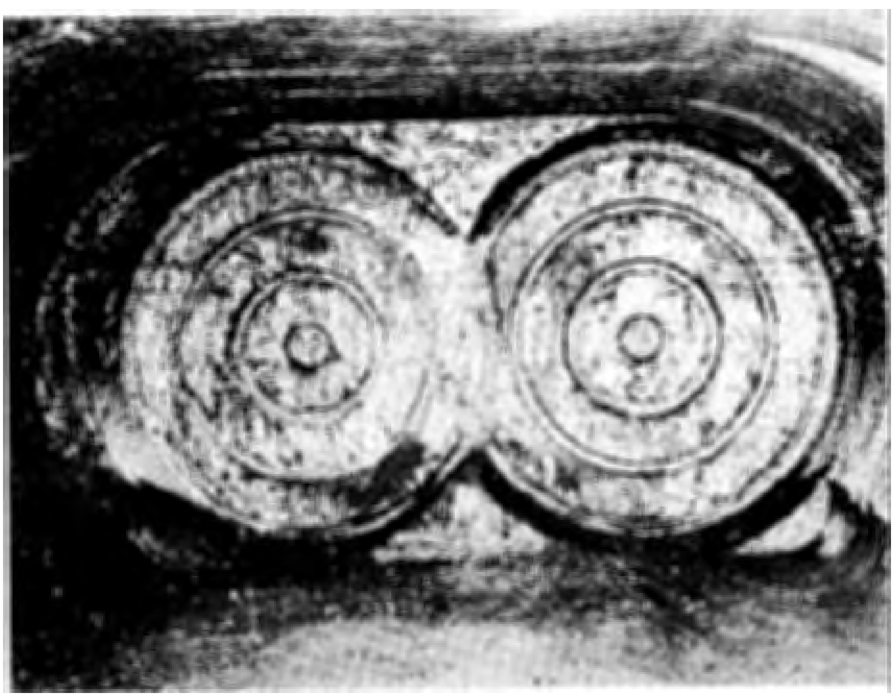

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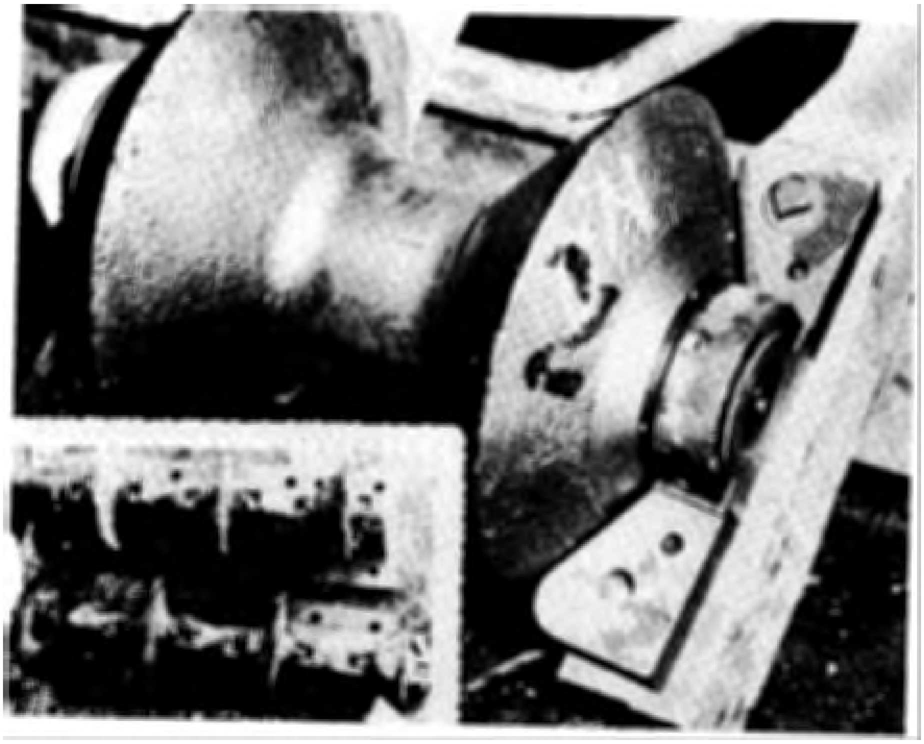

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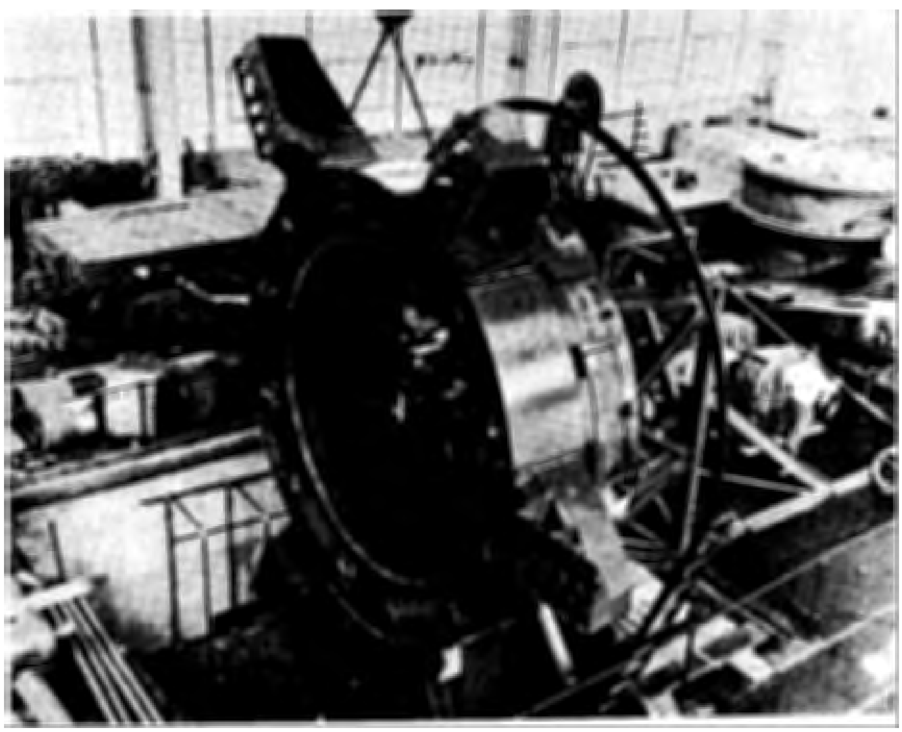

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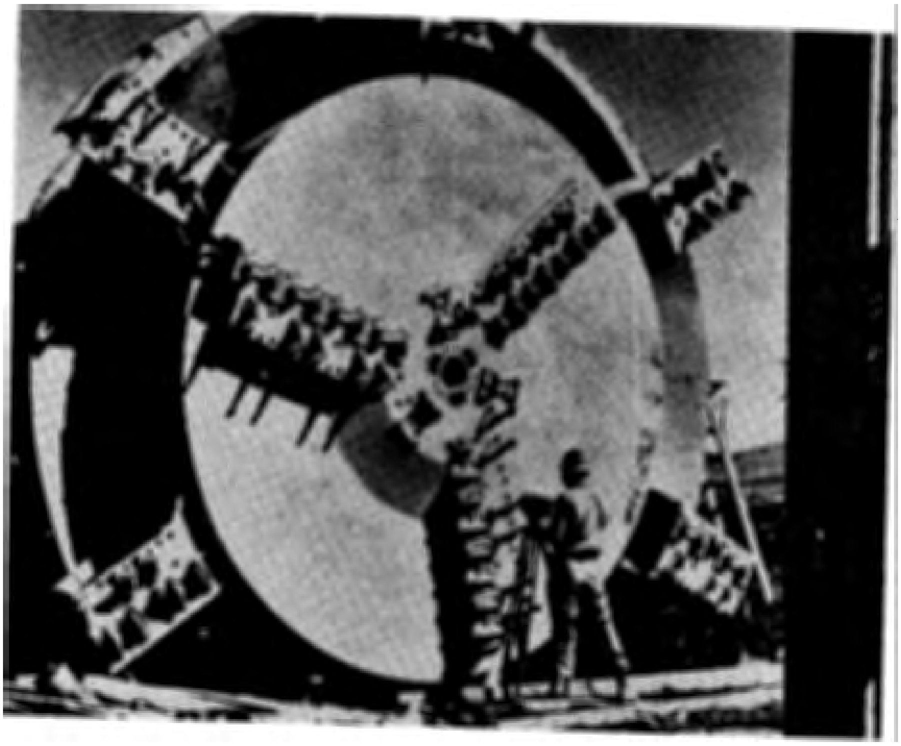

Fig. 11

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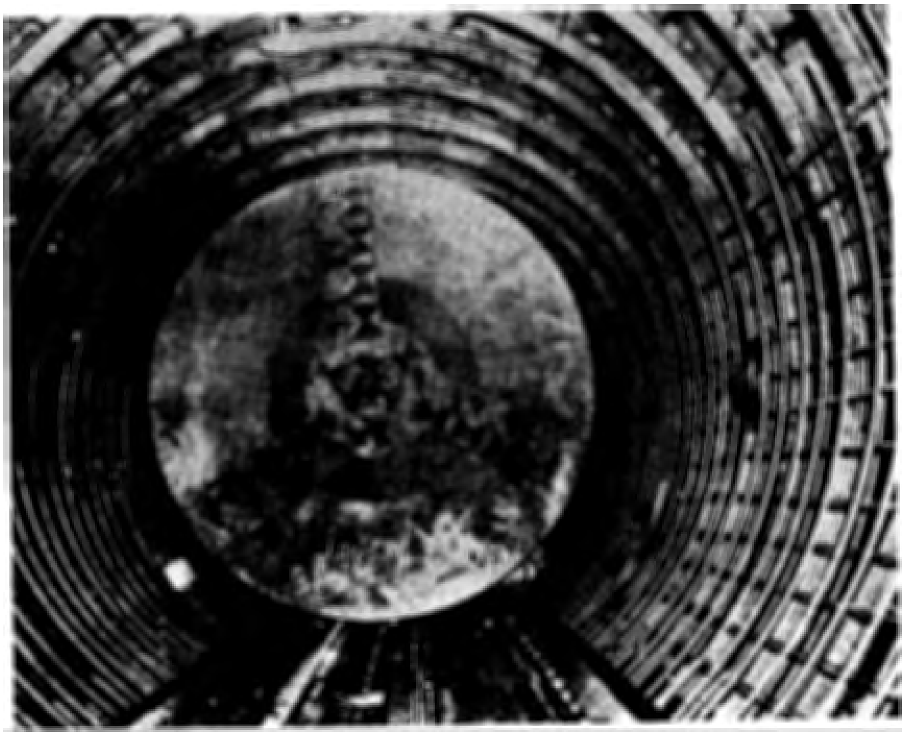

Fig. 13

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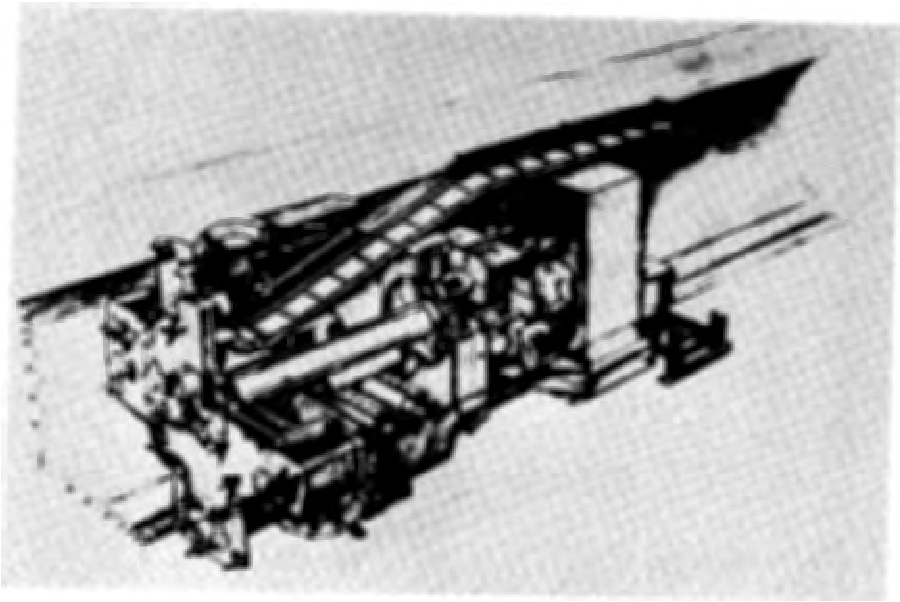


Fig. 15

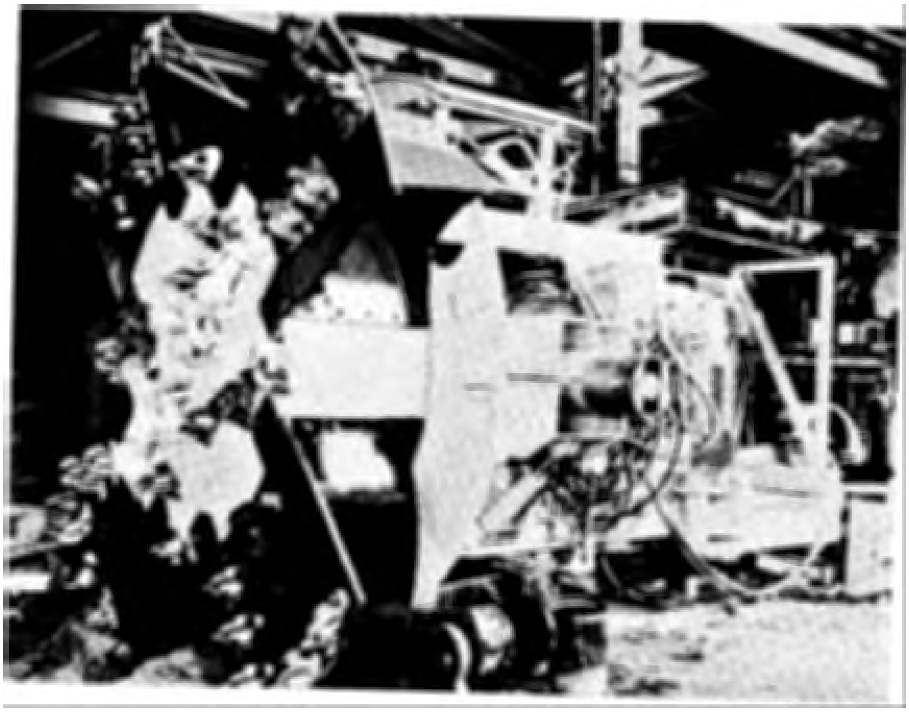
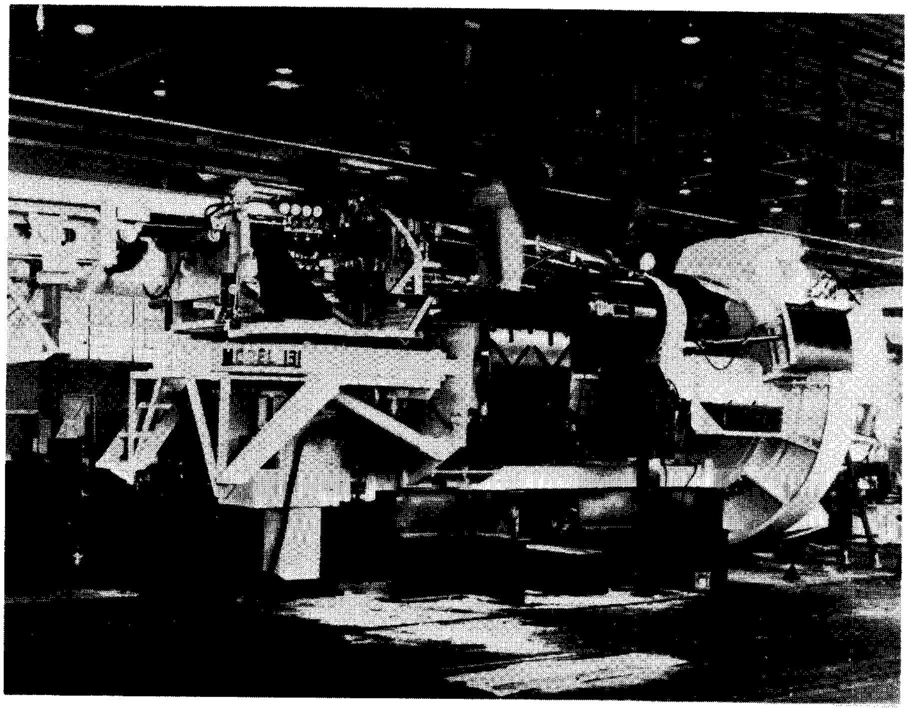


Fig. 16

Fig. 17

Fig. 18



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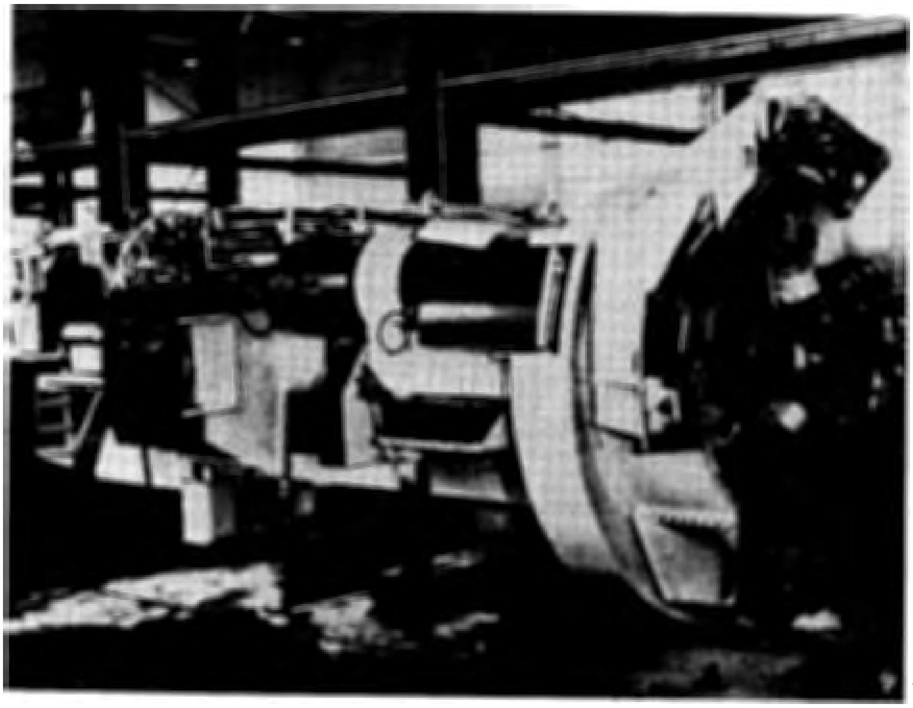
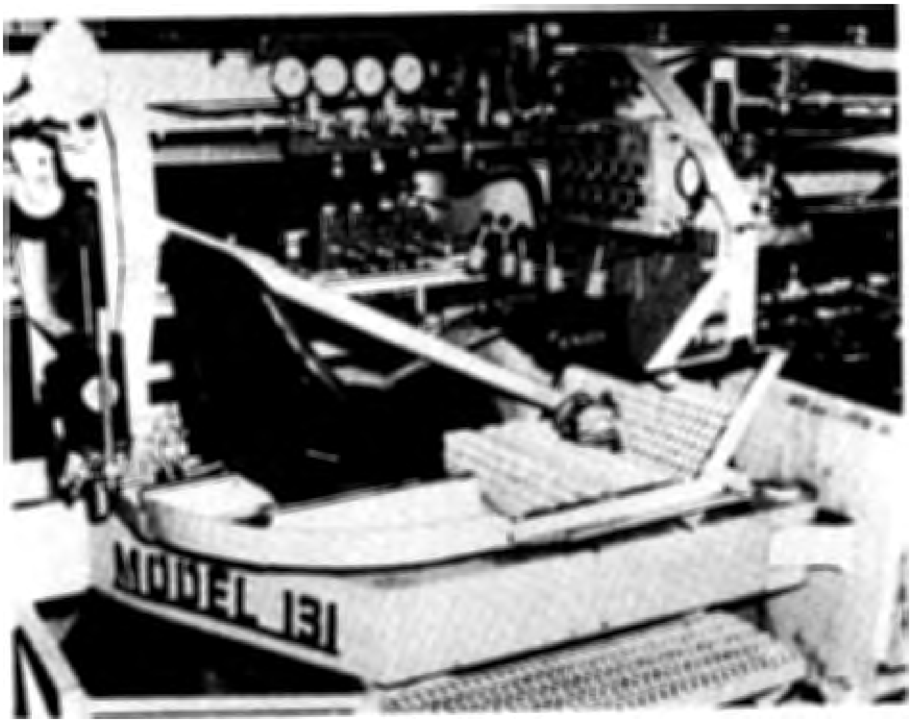


Fig. 20



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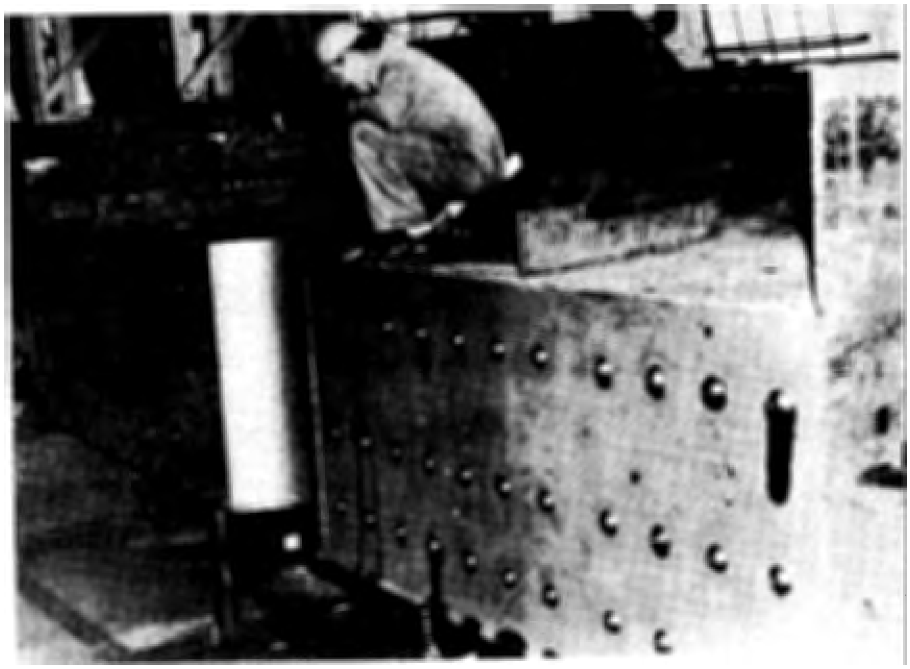


Fig. 25

Fig. 26

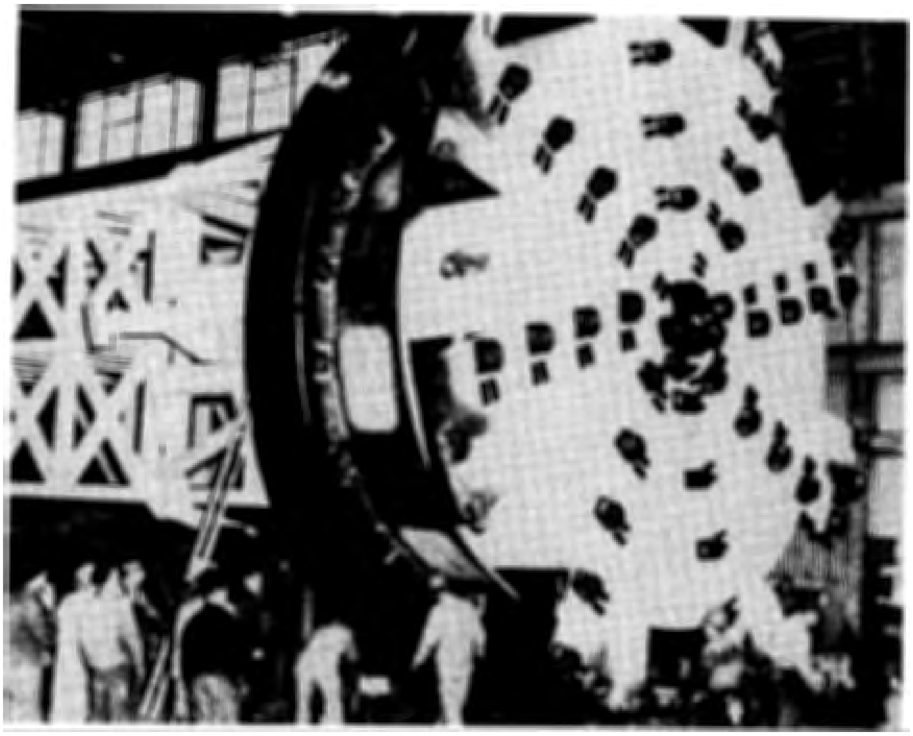


Fig. 27

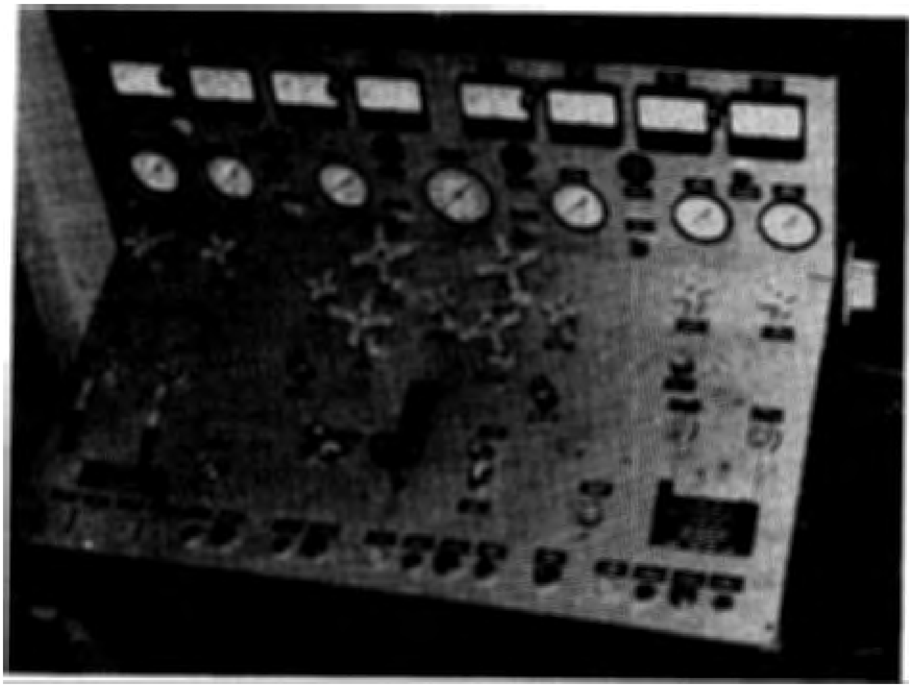


Fig. 28

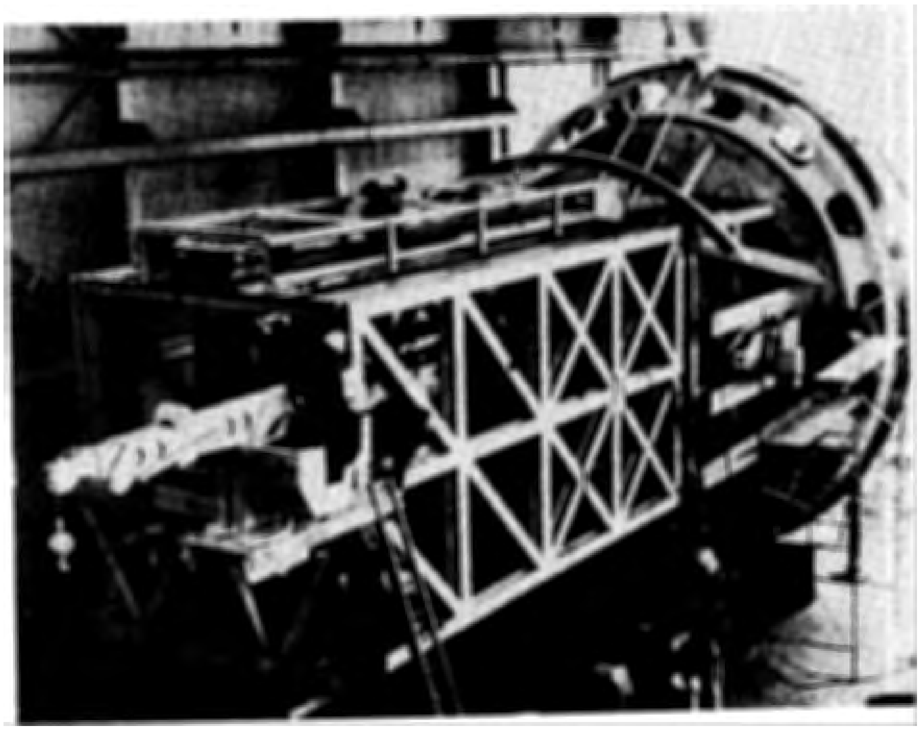


Fig. 29

ADAPTATION OF MINING METHODS

R. L. Loofbourow
Consulting Engineer

INTRODUCTION

Drilling and blasting are the conventional means of breaking rock in underground openings. The mining methods to be discussed in this paper have all been used on one job or another. The adaptations and combinations which will be suggested are selected from a great variety, as being particularly apt to meet the requirements of protective underground construction at low unit costs.

In the introduction to this session, your attention was directed to the diversity in the physical properties and conditions of rocks. The range in unit excavating costs is at least as great. Even in good, sound rock excavating costs can run as great as \$30 or \$40 in tunnels of small cross-section and as little as \$1 per cubic yard in quarries. Quarry methods cannot be brought directly underground, principally because (1) each foot of underground advance has to be broken out of a "solid," confined face of rock, (2) underground faces and working places are always more confined than their counterparts in quarries and (3) underground roofs and walls must usually be sufficiently stable for men and equipment to work safely beneath and very close to them. Underground methods which resemble quarrying to some degree are available and are capable of low-cost production.

DRILL ROUNDS IN HEADINGS

Underground openings such as tunnel headings are usually advanced in cycles of drilling, blasting and loading. Blast holes are drilled in "rounds" of holes, designed to break an increment of advance to the planned cross section.

Figure 1 shows a 36-hole round designed to break 7 ft in a hard, massive rock like granite. Numbers against the holes show the order in which they are to fire. This is called a "V-cut" round because the first holes, marked 1, 2, and 3, tear a wedge-shaped block of ground from the center of the face. Later holes progressively widen this initial opening.

Figure 2 shows a 50-hole round designed to break 15 ft in a similar hard massive rock. This is called a "burn-cut" round because the concentration of explosive is sufficient to shatter and eject the rock adjacent to a relatively large unloaded center hole, thus producing an initial opening in the shape of a rectangular prism.

Figure 3 shows a 38-hole, V-cut round designed to break 12 ft in a bedded limestone.

Study of drill rounds shows that (1) per foot of hole or pound of explosive, more yardage can be broken in headings of large cross section than in small ones, (2) per foot of hole, more yardage can be broken in a bedded rock than in a massive one, and (3) in large or small headings, whether in massive or bedded rock, some 25 per cent of the drilling and explosive are expended in making the initial opening or "cut," which breaks only 5 to 6 per cent of the yardage of the round.

It is common experience that larger chunks are produced in large headings where the hole spacing is greater than in small ones. Because work is concentrated, it is necessary that loading and hauling equipment be able to handle the largest chunks regularly produced. Use of the largest possible equipment eases this limit on the efficiency of drilling and blasting, but even in the largest headings it is usual to drill holes only 1.75 in. to about 2.5 in. diameter, and to space them 3 to 6 ft. Quarries commonly use 6- to 9-in. holes spaced 15 to 25 ft.

To keep equipment productively employed as much as possible, rather than moving into and out of working places, it is an advantage to break deep rounds. In the largest headings, 20 to 25 ft may be a practical limit.

Beds of strong sedimentary rock such as limestone may be separated by shaly partings. Where the strong beds are thin relative to the size of the opening, these natural weaknesses not only allow rounds to be broken with fewer holes and less explosive, but also tend to reduce the size of the chunks produced. Partings influence the shape of the opening for better or worse. Strong partings near the roof elevation may make it impractical to arch the roof, but if the partings are horizontal they may help to form a smooth, flat roof and floor. Fractures and cleavage in crystalline rocks help breaking, usually to a lesser degree, but are almost always irregular or at angles so that their effect on the shape of openings is generally bad.

BENCHING

Benching, illustrated in Fig. 4, is an adaptation of quarry methods to underground work. After an entry of suitable width has been driven, and its roof treated as may be needed for stability, the floor of the entry is drilled with vertical or inclined holes and blasted. By using coupled drill rods and hard metal bits, bench holes may be drilled effectively to depths as great as 100 ft. Benching affords the following advantages:

1. drills may work far enough back on the bench that they do not need to be moved for blasting;

2. shovels and trucks work without interference from other equipment and without interruption, except that shovels must be moved back for blasting;

3. good ventilation can be maintained in long rooms without the use of ducts, small blowers or doors in haulage ways; and

4. to the limit set by the capacity of loading and hauling equipment to handle large rock, the drill pattern may be varied for greatest efficiency.

LONGHOLE STOPING

This method, illustrated by a diagrammatic cross section on Fig. 5, is another underground mining method used in strong rock. It has some of the advantages of quarrying. Preparatory work consists of undercutting the block to be mined with a series of inverted cones or funnels through which the broken rock is drawn for loading, and in which large chunks may be blasted. Long blastholes are drilled in vertical rings from drilling drifts on the edges of the block to be mined. A vertical "slot," not shown in the illustration, is opened over the full cross section of the stope and forms a large "free face." Retreating from the slot, vertical slices are peeled off by successively blasting rings of longholes. Broken rock falls into the draw points at the base of the stope. Note that all work is done in small openings. No one needs to enter the large stope. Blastholes are carefully aligned, but no trimming or other stabilizing is done.

The amount of preparatory work required makes longhole stoping unattractive for openings less than about 150 ft high. Where strong, tight rock extends for several hundred feet vertically, openings of this type should be useful for storage of fluids. Such storage could be at high pressure if depth were chosen appropriately.

PARALLEL, ADJACENT HEADINGS

In a single tunnel heading, all effort is devoted to a single purpose, to advance the heading. Much of the equipment and often some labor stands by through parts of the cycle of drilling, blasting, and loading. Each of these main steps includes other chores such as trimming loose rock and cleaning up after blasting, which are rushed or overlapped with other work.

If a number of parallel, adjacent headings are worked, as shown by Fig. 6, each step may be performed more efficiently. Adequate ventilation can be provided more quickly and at less expense to a number of parallel adjacent headings than to the same number of single, isolated headings. Under such a plan, the two relatively narrow entries near the top and bottom of the illustration would be driven first and then the parallel rooms between them. Access, haulage and services would be supplied through the entries.

PILOT HOLES

In his paper, "Large Diameter Boring," Mr. Haspert told you of boring holes 2, 3 or 4 ft in diameter, rapidly and at reasonable unit costs even in hard rock. In this paper it was shown that it costs five or six times as much to break a unit of rock from an initial opening or "cut" as to break the remainder of a heading round. Referring again to Fig. 6, consider the possibility of boring pilot holes between the two entries, more or less on the center line of each room. Pilots for the first group of rooms could be drilled from small "turn-offs" made as one of the entries is advanced. The pilots would serve as a prefabricated "cut." If they were placed near the roof, it should be possible to keep them open for rapidly removing powder fumes after blasting.

SMOOTH BREAKING

Figure 7 is a grim example of the ragged, shattered roof and walls which may result from ordinary blasting in a rock with cleavage and some jointing. The illustration shows a draw point under a longhole stope. The original irregularities have undoubtedly been accentuated by concussion from blasting chunks brought down from the stope.

Figure 8 shows almost plane roof and floor formed by carefully mining to regular partings in limestone beds. It may be noted that the walls tend to slab, perhaps as the result of pressure imparted by 2300 ft of overlying rock. Incipient slabs may be seen near the center of the left-hand wall.

It is difficult or impossible to make entirely satisfactory surfaces in rock which has strong, irregular defects, but gratifying results can be achieved in some circumstances. The tunnel of Fig. 9 is in greywacke with a strong cleavage nearly vertical and at right angles to the axis. The irregular left wall and roof are part of the original tunnel driven conventionally. The right wall has been widened by blasting horizontal longholes, parts of which may be seen almost continuously along the wall.

The astonishingly smooth walls shown in Fig. 10 are the result of widening a tunnel in massive granitic rock by blasting horizontal longholes. Work was done with extraordinary care for the Corps of Engineers, who set up specifications to obtain minimal overbreak.

In outlet tunnels for Swedish underground power plants smooth, stable walls have been made by blasting carefully aligned, comparatively long holes, with explosive charges reported to be as light as 0.1 pound of explosive per linear foot of hole.

APPROXIMATE COST OF EXCAVATION AND MINIMAL HAULING

Accurate estimates of the cost of a specific job can be made only by detailed work based on known conditions on that job. It has been stated that the range of natural conditions alone is large.

Figure 11 is presented as a rule-of-thumb guide. It also indicates the cost differences which may be expected in the types of excavation which have been discussed, in two types of sound rock. It is to be emphasized that costs in these ranges are not to be expected except where a large volume of uniform work is to be done. The unit costs of excavation in entrances and in openings of special size, shape or location would be expected to exceed the costs indicated, in many cases greatly. Care has been taken to show in some detail on the illustration, the conditions under which the costs are considered applicable.

These data are based on analogies to several types of mining in hard rock and in limestone with allowance for somewhat higher mobilization, equipment, and administrative costs and more allowance for contingencies than is needed on mining operations of very much longer life.

HOISTING, CRUSHING AND CONVEYING

Observation of quarrying and appropriate types of mining shows that, except in rock with closely spaced defects, efficient excavating produces many large chunks. In the preceding discussion, it has been assumed that rock is to be dumped at approximately the mining elevation and that trucks would be used over a comparatively short, nearly horizontal haulage-way. If work is to be done in a nearly level country, rock must be lifted by one means or another from whatever depth is required for protection or to reach a favorable rock mass in which to excavate.

Entrances on gentle slopes are advantageous for dispersal and for the ease with which ordinary vehicles may enter or leave. Off-highway trucks haul with reasonable efficiency on grades up to 10 per cent and with less efficiency to 15 or 20 per cent. For work of only moderate volume, or for extensive work at a depth of only a few hundred feet, speed and simplicity favor the use of trucks alone.

If work is at greater depth, or if a very large volume of rock is to be moved, rock might be hauled on the level to one or more underground points and raised to the surface in skips through vertical or inclined shafts, or by belt conveyors which operate with high efficiency and moderate operating cost at grades up to 30 per cent. Except possibly in inclined shafts, rock broken by the means discussed here would have to be crushed to be handled efficiently in presently used types of skips or belts.

By rule-of-thumb, coarse, hard to moderately hard rock can be crushed and hoisted about 1000 ft vertically for \$0.30 to \$0.60 per cubic yard, whether in vertical shafts or on belts, including all operating costs and writeoff, but not including the cost of the shafts or inclines. If work is hurried to completion over a comparatively short period, cost is likely to be nearer the higher figure. As between hoisting vertically or on belts, the latter are likely to be favored where cost of power,

power installation and labor are high.

EXAMPLES OF APPLICABLE METHODS AND EQUIPMENT

Figure 12 is a diagram of the U. S. Bureau of Mines Experimental Oil Shale Mine near Rifle, Colorado. Pillars are about 60 ft square and rooms are of the same width. The top headings are about 27 ft high. The floor is lowered in one or ultimately two lifts of about 23 ft each. Figure 13 shows a four-machine, two-man jumbo used to drill heading rounds designed to break 15 ft. This marlstone has regular parting planes which, with careful mining, accounts for the smooth roof and floor. Figure 14 shows much of the other equipment used.

Figure 15 is a cross section of the recent Stornorrfors Power Plant Outlet Tunnel in Sweden. The bottom is lowered by benching in two lifts. Two heading jumbos on this work are illustrated in Fig. 16. A battery of 12 benching drills is seen in Fig. 17. This also shows a nicely arched and comparatively smooth roof.

FIGURES

1. 36-hole round for massive hard rock, The Blaster's Handbook, copyright, E. I. du Pont de Nemours & Co., used by permission.

2. 156 sq. ft. tunnel, 4-5/16 cylinder-cut, Swedish State Power Board.

3. 38-hole round, 12 ft. deep, used in bedded limestone, Tri-State Zinc Co., Ill., Reprinted from U. S. Bureau of Mines Information Circular 7730.

4. Diagram to show the use of benching to lower the floor of a tunnel.

5. Vertical cross section of a longhole stope showing drilling pattern.

6. Parallel rooms between two entries--dimensions appropriate to requirements and rock.

7. Opening of about 8' x 6' showing rough walls formed by ordinary blasting in a hard, crystalline rock (amphibolite) having cleavage inclined about 20 degrees from horizontal.

8. Long room, about 32' x 46' in bedded limestone, Barberton, Ohio. Note plane roof and floor formed on limestone partings, Pittsburgh Plate Glass Co.

9. Right wall broken by longholes drilled from slot opposite rear of loader; left wall and roof are parts of original tunnel driven conventionally. Highway tunnel enlargement near Valdez, Alaska, Gates and Fox Co.

10. Tunnel in granitic rock showing clean walls broken by longholes drilled from a slot, the remnant of which is seen as a ring. Buford Dam, Georgia, Gates and Fox Co.

11. Approximate total cost of excavating and minimal hauling.

12. Diagram, U. S. Bureau of Mines Experimental Oil Shale Mine, Rifle, Colorado, Reprinted from U. S. Bureau of Mines Report of Investigation 4739.

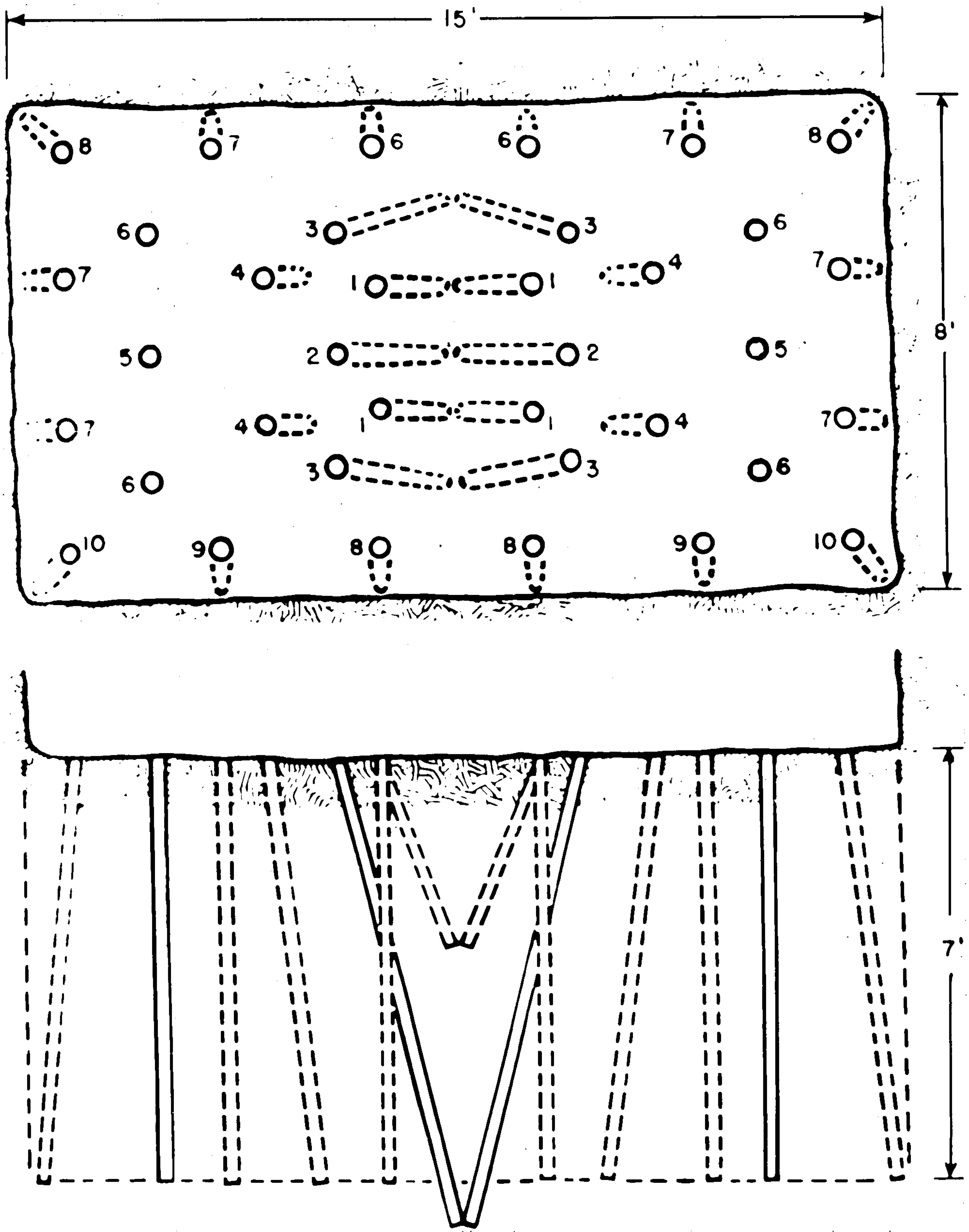
13. Heading Jumbo with 4 percussion drills, each adjustable as to height and angle. Oil Shale (Marlstone) Project, Rifle, Colorado.

14. Foreground: 3 yard electric shovel
    15 ton Euclid
    On Bench
    (background): Portable compressor-receiver unit, electric benching Jumbo with 4 percussion drills
    Oil Shale (Marlstone) Project at Rifle, Colorado.

15. Cross section of the Stornorrfors tail-race tunnel, Swedish State Power Board.

16. Drilling platform in a 1720 sq. ft. tunnel, Swedish State Power Board.

17. Drilling apparatus in tunnel, Swedish State Power Board, 1958.



36 Hole Round, 7 ft. deep, for massive hard rock
DuPont Blasters' Handbook

Fig. 1

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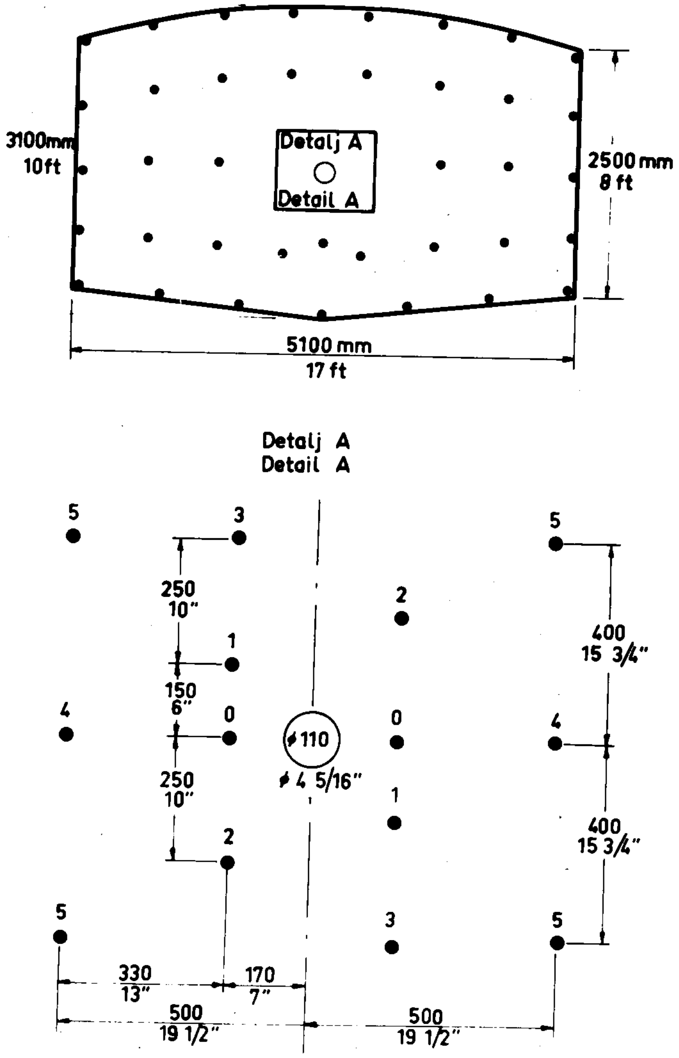


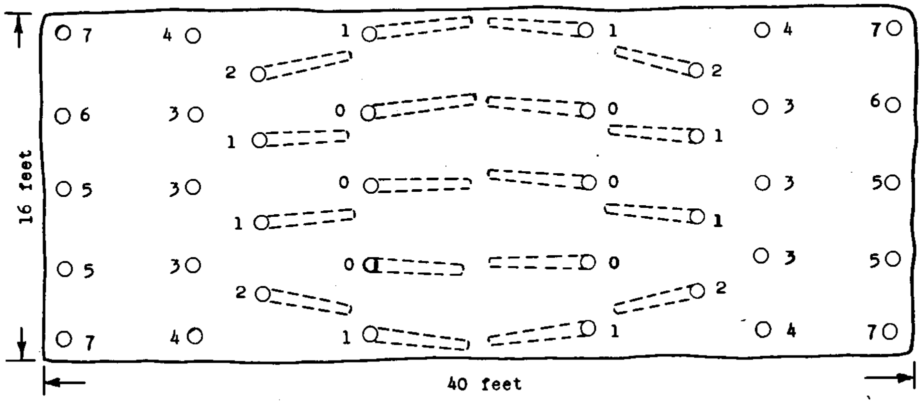
Håldjup	m	4,8
Indrift per salva	m	4,7
Antal	fm^3	68
Kilhål	st	14
Strosshål	st	36
S:a hål	st	50
Borrmeter		240
Bm/m^3		3,5

Depth of hole,	ft	15'9"
Advance per round,	ft	15'5"
Number of cu.yd. of rock		89
" " cut holes		14
" " stoping holes		36
" " holes, total		50
Drilled footage per round		787
- " - per cu.yd.		8,8

14,5 m^2 tunnel. 110 mm cylinderkil, Täby

156 sq.ft. tunnel. 4 5/16 cylinder-cut, Täby

Fig. 2



38 Hole Round, 12' Deep, used in Bedded Limestone
Tri-State Zinc Co., Jo Daviess Co., Ill.
U. S. Bureau of Mines Inf. Circ. 7730

Fig. 3

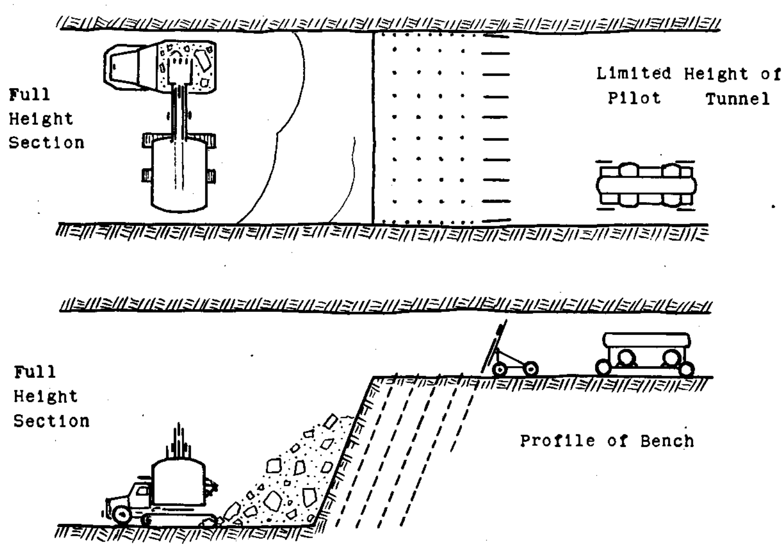
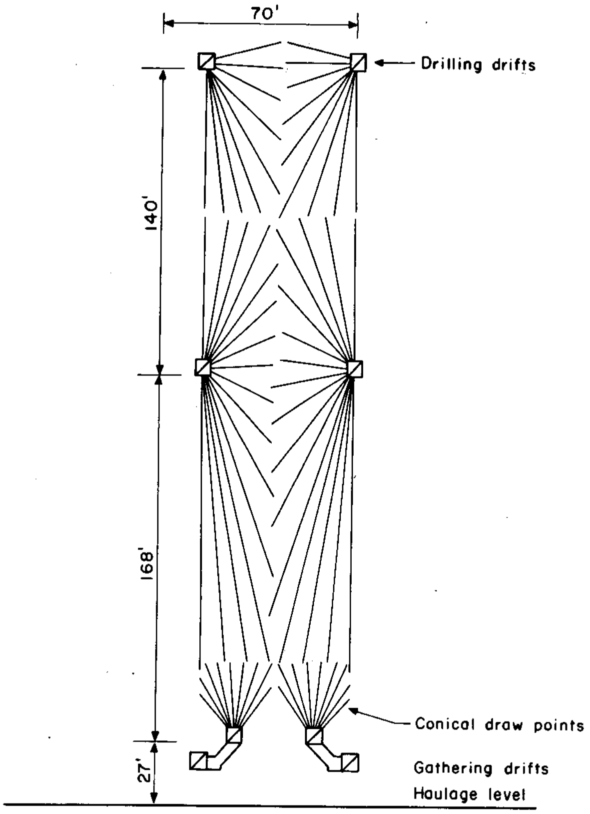


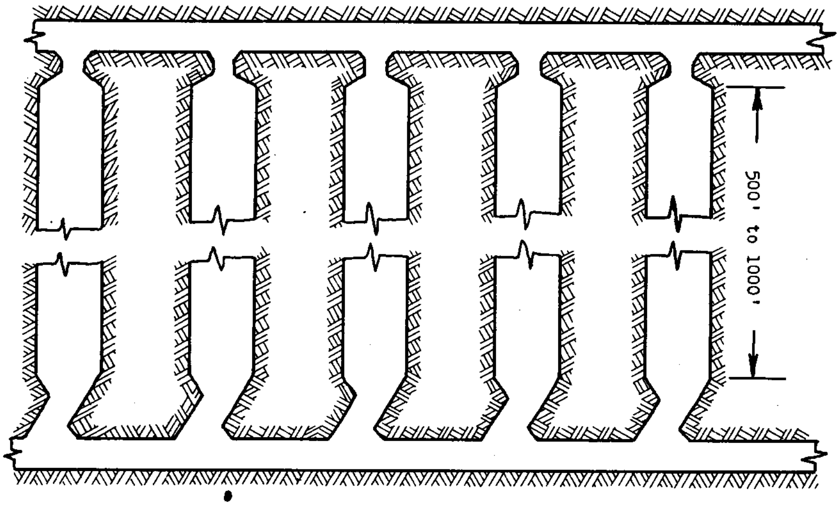
Diagram to Show the Use of
BENCHING TO LOWER THE FLOOR OF A TUNNEL

Fig. 4



Vertical cross section of a longhole stope
showing drilling pattern

Fig. 5



Parallel Rooms between Two Entries
Dimensions appropriate to requirements and rock

Fig. 6

Fig. 7

Fig 8

Fig. 9

Fig. 10

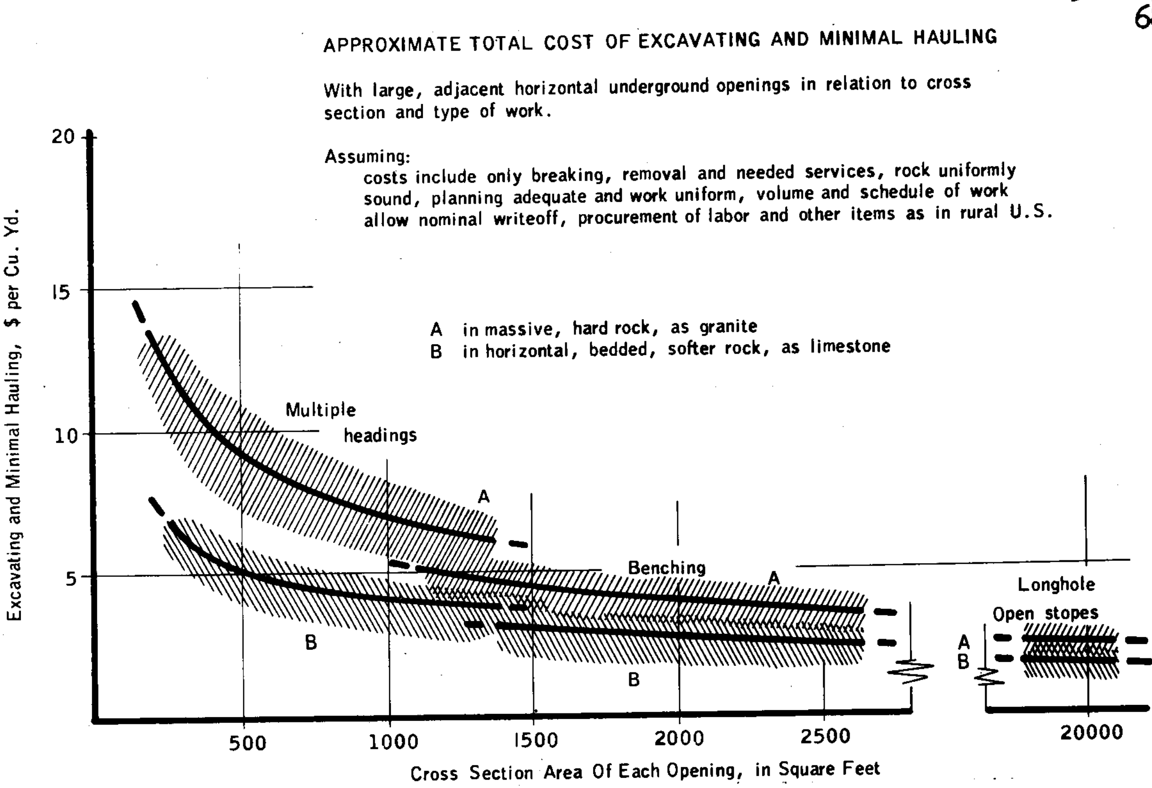


Fig. 11

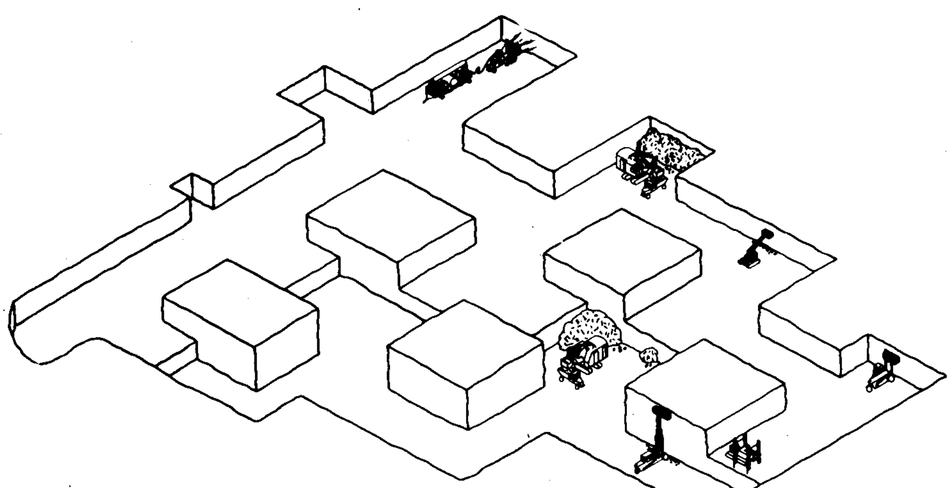


Diagram
U. S. Bureau of Mines Experimental Oil Shale Mine
Rifle, Colorado
From U. S. B. M. Inf. Circ. 4739

Fig. 12

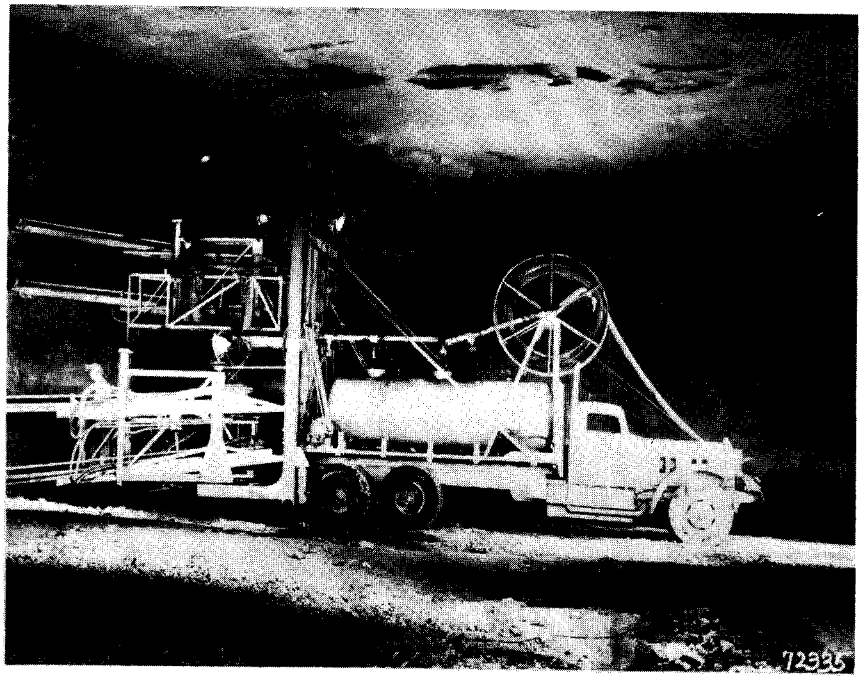
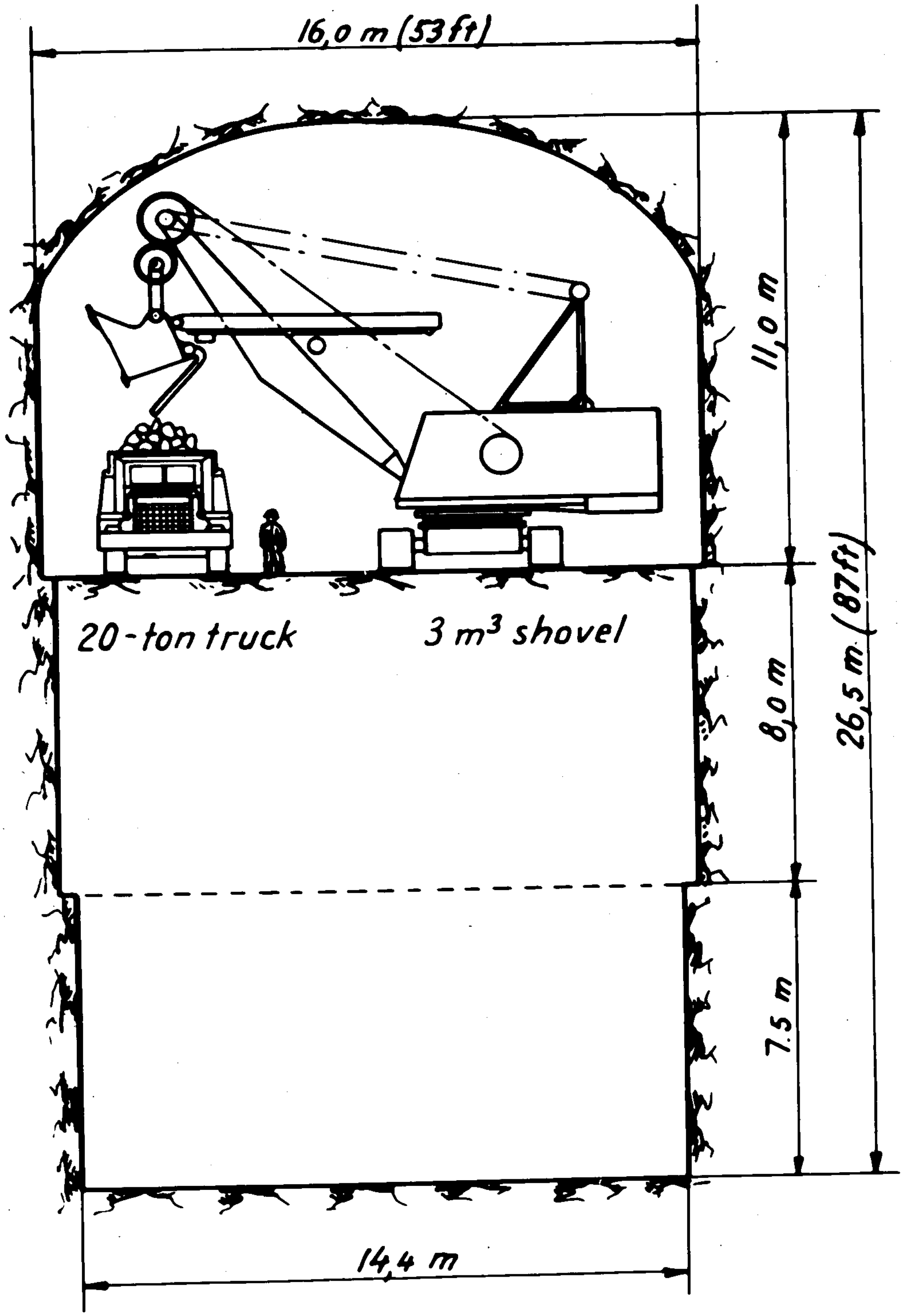


Fig. 13

Fig. 14



CROSS SECTION OF
THE STORNORRFORS TAIL-RACE TUNNEL

Fig. 15

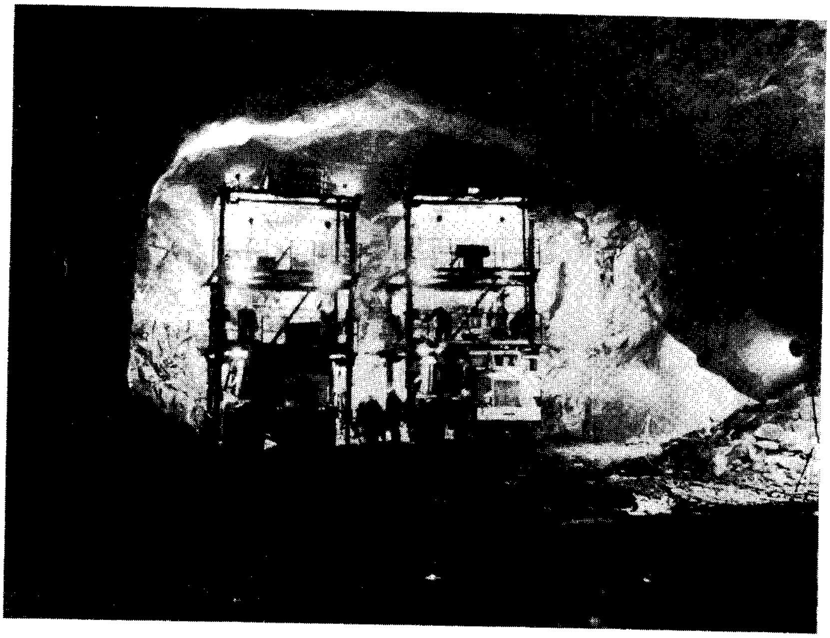


Fig. 16

Fig. 17

HIGH-SPEED, LOW-COST EXCAVATION METHODS

J. J. Walsh and Robert Budd
Walsh Construction Company

We have been asked by The RAND Corporation to discuss advantages of flat seam mining as applied to protective construction and solid rock. The problem stated to us was to devise a quick and economical method of construction which will afford maximum protection for equipment, ample and expandable floor area, will require a minimum of roof support, and will require a minimum of interior structure for adequate housing of personnel and equipment.

To obtain maximum protection, the choice of a suitable rock formation is, of course, all-important. We must assume that such a choice has been made after consideration of the necessities as evidenced by the tests that have been performed to date. We are to understand that this protective construction may be somewhat on an emergency basis. For that reason, a design should be selected which will be subject to some variation after construction is commenced.

We have considered more than tunnel practice or mining practice. We feel that the miners in the flat seam beds have pointed a way to a most economical method of construction of this type of proposed structure. A miner must dig where the ore is, and this often means in weak formations. If the methods that he uses in the weak formations are transferred to thoroughly competent strata, we feel that roof support should be held to an absolute minimum.

Since the diesel engine has become commonly used, and since the advent of rubber-tired mining equipment, many of the mines have gone to this type of equipment. Some examples would be the lead mining by the St. Joseph Lead Company; lead-zinc mining in the tri-state district; copper in White Pines by the White Pines Copper Company; limestone by Pittsburgh Plate Glass, United States Steel, and many others; iron ore by the Hanna Company of Iron Mountain, Missouri; potash by U. S. Potash, International Chemical, and Potash Company of America. U. S. Gypsum in many locations is mining this way. Also, there are the Pacific Coast Borax Company, International Salt, and many others.

We would estimate that the companies listed must be producing to stockpile for an average of $1.50 to $2.00 per ton of ore. If that were not so, the mines would become uneconomical and closed. For instance, let's take the White Pines Copper Company where the yield is 18 lbs of copper per ton of rock removed from the mine. You simply could not live at that price unless you had a very cheap operation. This is certainly true as applied to salts and limestone where the sale value of the material on the open market is only about $1.50 to $2.25 a ton, depending on location and the competitive situation. I bring that in to establish costs and to give you some background.

The cost of an undesigned structure in an unspecified strata at an unspecified location is somewhat theoretical. But when we go back to what the mines are doing, we do have a cost basis, of course.

Figure 1 is a really horrible example, and I show it just for that purpose. This is one of the tunnels at Niagara in which my company was interested. The tunnel is 54 ft wide. The strata is thoroughly incompetent. We have to face the fact that the designer of a tunnel has little choice. The tunnel must go from one point to another. All he can do is bury that tunnel in depth to get the safest roof structure possible, but his alignment cannot be changed.

In Fig. 2 we see a modern flat seam mine. The roof is flat solely because this is a limestone formation, and the flat roof is inherent. The roof could be arched; and if it were, it would probably be arched due to the inherent break, a characteristic advantage. These mines use high-speed heavy-duty equipment and develop multiple faces as quickly as possible.

Figure 3 shows a shovel loading and heavy haulage equipment.

Figure 4 is a roof-bolting jumbo. It is readily movable from place to place. However, this figure points up the fact that too high an excavation becomes costly because of the difficulty of reaching the roof for scaling and other purposes.

Figure 5 shows the drilling equipment mounted on trucks. This is simply a quick sketch demonstrating how all equipment is designed to be movable.

Figure 6 is a schematic layout of a checkerboard block system of mining. It makes no pretense of being a finished product. This is an area which we planned to excavate between 16 and 20 ft. I feel that 20 ft is approaching the maximum for one-pass flat seam mining. When you go above that height, the drilling equipment, scaling equipment, etc., become rather topheavy and hard to move about where the pillars are twice the width of the galleries or roadways. This is simply a matter of proportion.

First of all, we must have access. We might propose two tunnels and then indicate a shaft. We could have a third entry; we could group the three entries together. We could do many things; but please assume that you are going to want some dispersal of access, so we are showing the access tunnels at widely divergent points.

The first move would be to sink the shaft and drive these tunnels in from the outside to the shaft to establish ventilation. After that is done, we would progress as shown in Fig. 7. Now, extraction commences. We can blast at several locations, and we have many working faces. We would put in one team of men with a shovel, a scaling jumbo, roof-bolting jumbo, or whatever we required. Some of the jumbos may serve dual purposes. It would be very simple from this time on to blast two faces on a shift and to load out two faces. Instead of the usual tunnel cycle of drill, blast, and muck, we have men drilling at some locations, and they drill all during a shift. We have men loading, and they load all shift. The haulage crew hauls all shift. At the end of the shift, the face is prepared for blasting. The blast gasses return to the ventilation shaft and are carried out.

In Fig. 8 we now have ten faces on one line. If we want speed, we can put in double units. In other words, we can put two crews on one side, two crews on the other side. That would be very simple. We are primarily interested in speed. In the area shown there, we assume that the square pillars are 60 ft and that the entries or haulage ways or rooms, whatever you choose to call them, are 20 ft wide. The whole area is 20 ft high. You would have 521,000 ft^2 of available floor space in the 30-ft drifts. That is the equivalent of 17,330 ft of tunnel, 30 ft wide by 20 ft high. If we were to blast 12 ft to an advance, we would have 1460 advances to make. If we average two shots on one side and two shots on the other side each shift, and work two shifts spaced to allow ventilation time between the shifts, we would get eight advances in each 24-hour day. It would require approximately 182 days to excavate the area, once excavation and development were established.

As I mentioned before, this rate could be very easily increased by the addition of more working teams. Perhaps we could easily average five or six faces per side per shift. That gives you some idea of the speed.

Now, we have access tunnels. How fast can we drive those? There again, we would really like to know a little more about what we are driving in and what we are driving from. A thousand feet a month is not irrational to discuss for the speed of those drifts. To be conservative, I think we can make 800 ft a month. A great deal of this is dependent upon the particular problem.

What are we netting? If we are going to work five days a week, we pay for 40 hours; if six days a week, our costs go up 16-1/3 per cent; seven days a week, our cost then advances to 33-1/3 per cent of additional labor. We have to decide at what rate we want to operate before we can intelligently discuss costs.

At the beginning I gave you an indication of what the mines are able to accomplish in terms of production costs. We have to face the fact that for protective shelters, our shifts have to be retired against far less volume than a mine will produce. A mine will be in operation for ten or fifteen years, so that the write-off can be absorbed.

Also, when you go into a thing of this nature, you are going to have to provide electric power. You are going to have to provide water. You are going to have to provide housing for some of your men in some localities. You are going to have to provide office space, shop space, and ventilation. That is not a small thing. We've got to provide all those, and we have a short quantity to write off on. Now, that may seem discouraging; but I still think we have developed the most economic method of putting in the underground structure.

Now, I want to return to ventilation for a moment. To get high-speed operation, you've got to use diesels which produce diesel fumes. If you do a lot of blasting, the powder fumes are considerable. It doesn't seem possible until you sit down and figure it out, but you will probably move more tons of air per day into that area than you move out rock.

FIGURES

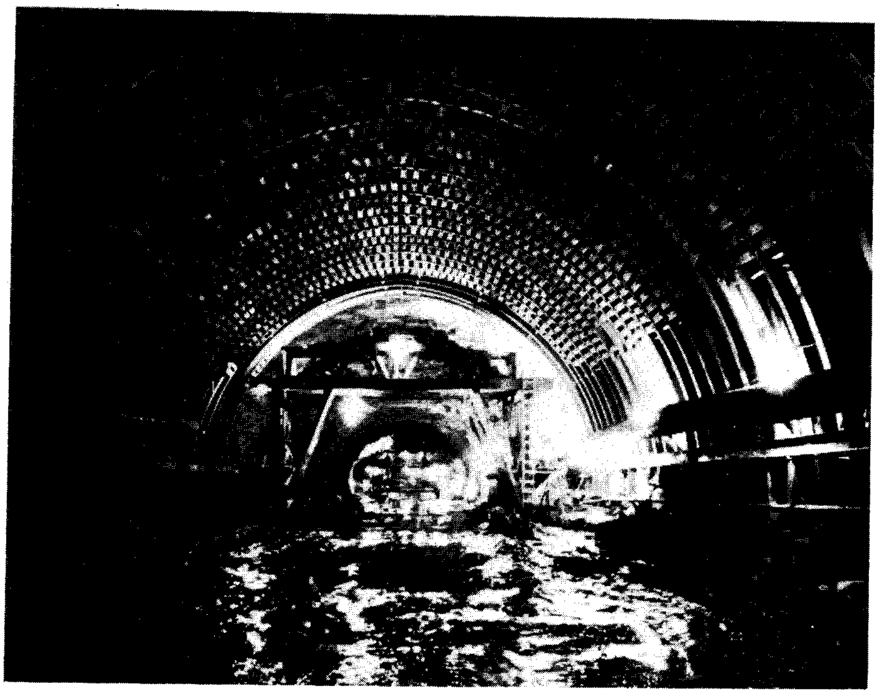


Fig. 1

Fig. 2

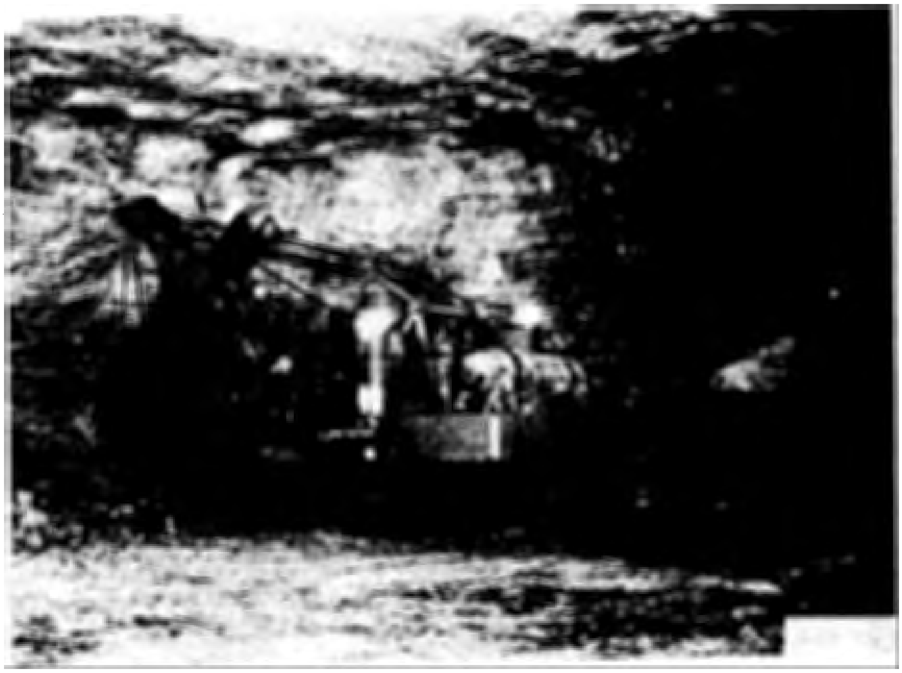
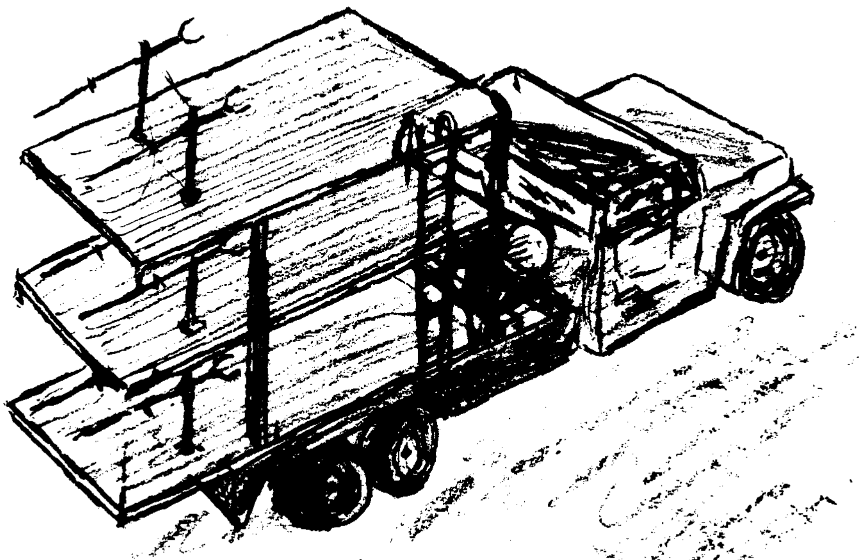


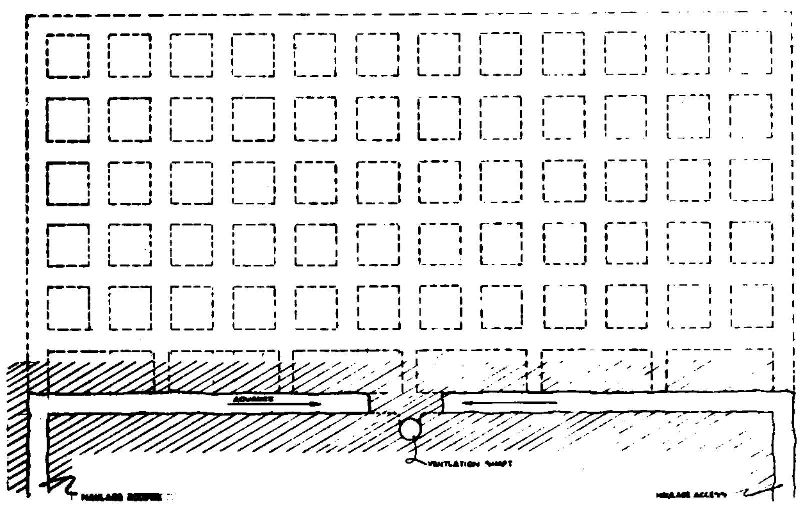
Fig. 3

Fig. 4



TRUCK JUMBO

Fig. 5



STAGE ONE

Fig. 6

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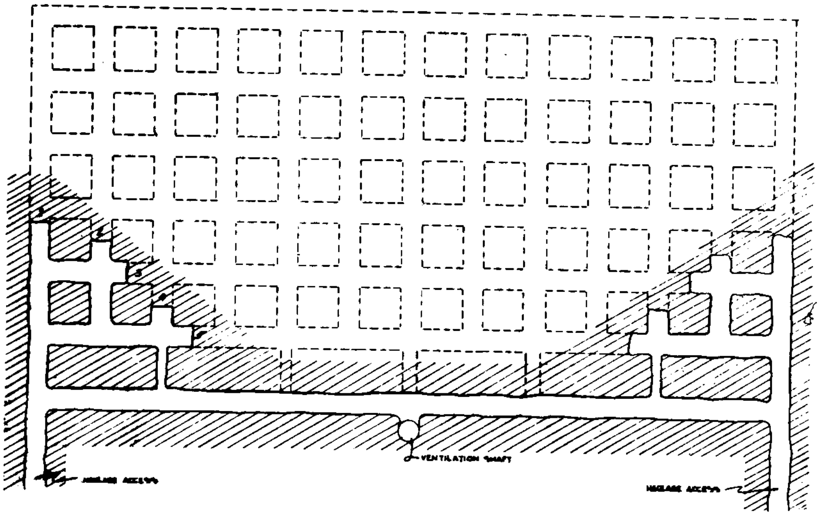
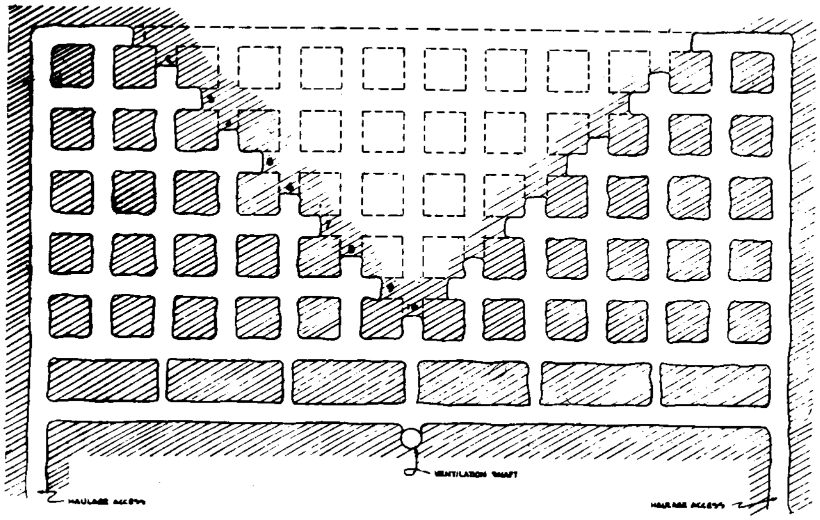


STAGE TWO

Fig. 7



STAGE THREE

NOTE:

1. [illegible]
2. [illegible]
3. [illegible]
4. [illegible] VENTILATION [illegible]
5. [illegible]

Fig. 8

UNDERGROUND SHELTER STUDY

Edward Landway
Luria Construction Corporation

The Luria Engineering Company for many years has been engaged in the fabrication and erection of standard and especially designed structures for the military forces. These structures have ranged from our standard line of buildings to specially adapted aircraft readiness and maintenance hangars and docks.

Because of our ability to adapt structures to special situations, we were requested by The RAND Corporation to investigate the possibility of erecting a sound and economical structure within certain underground areas. These areas were defined as hard-rock tunnels with a basic configuration in sections of 30 ft in width and 16 ft in height, but including an alternate section of 30 ft in width and 32 ft in height. The basic tunnel would contain a one-story building and the alternate would contain a two-story building.

Criteria for the design of these buildings would be:

1. That they be moisture-proof and dust-proof.

2. That the exterior steel construction would be of such a nature that alkali or acid filtrating waters would not harm or deteriorate the metal.

3. That the interior would have an insulated and acoustic treatment which would allow temperature control and humidifying of necessary areas but would give acoustic value to the over-all structure. The interior treatment would be of a varied colored arrangement that would have a pleasant effect on the people working within these buildings.

Our first approach to this problem was to try an adaptation of our standard rigid frame structure within the defined areas, leaving a clearance of approximately two feet to rock line from the outer building dimensions. After considering the many problems that would be involved in the erection of such a structure, and particularly in the design of a suitable panel, framing into the structural members, which would meet the criteria previously mentioned, it was decided that the erection costs of such a structure would be prohibitive due to the necessity of erecting it from the inside.

We therefore decided that a new approach was necessary to not only meet the criteria presented to us but also to give a completely flexible structure which could be erected in any given situation and which could be made to conform to the design requirements for additional space, in any direction. The design we finally accepted as meeting all of the necessary criteria and also being the most economical, both from a fabrication and erection viewpoint, is a protected metal insulated panel, 1 ft in width, and welded to form a rigid frame structure. This frame structure, in order to insure stability, has a tie-rod from eave point to eave point at intervals of 6 ft. Each panel is insulated with a one-inch fiber glass batten snapped into place between the panel flanges. Erection of panel to panel is accomplished by through-bolting with a neoprene rubber seal between both panel flanges. This is continuous from the base to the eaves and over the roof section of each panel. Interior and acoustic treatment is accomplished by then attaching a perforated hardboard wall and ceiling to the interior of the panel flanges. This hardboard interior then can be painted any designated color, resulting in a pleasant and attractive working area.

This particular metal panel when completely bolted through, forming a rigid frame, is capable of maintaining a live-load of 15 lb/ft^2. It must be borne in mind that this structure is extremely economical and therefore is not adapted to blast conditions. It has the advantage over a concrete structure of being completely free of the surrounding rock if desired. We have worked out a method of isolating the side walls and roof from the floor through use of a rubber pad and spring-loaded anchor bolts.

It is our belief that this basic structure not only meets all criteria established, but as previously pointed out, is extremely economical with high speed of fabrication and erection. In order to be specific as to the basic, or one-story structure, we pour a concrete floor on a cushioning base of sand or gravel, including side angles to which our panel structure is anchored as previously described. This structure will be erected by setting up a fabricating area where we will weld and bolt these panels together in increments of six. Each increment will be brought to place by a method which we have designed, and erected in plase to form an increment of the building. We have provided methods of meeting various right-angle and other angular variations as well as transitions in height. Our investigations have shown that it will be possible to fabricate and erect in place this one-story building for a cost of \$6.50 per ft^2. The two-story building will be constructed by essentially the same methods except for the fact that the second story floor area will be poured and the side panels to the second floor will be erected in multiples and the second story panels brought into place by a method we have worked out to form the sides and roof of the second story. The floor loading for the second floor can be varied for the installation of special machinery or for other applications. We have estimated the cost of this two-story building at $12.00 per ft^2 for the actual area of the first floor or $6.00 per ft^2 per floor. These structures have been estimated on the basis of 200,000 ft^2 of habitable area for the one-story building and 100,000 ft^2 for each floor for the two-story building.

These structures can be speedily erected depending on, of course, area allowed by the excavating contractor. This is a gallery operation as far as hard rock mining goes, and it will be necessary to schedule our erection operations so that we offer the least possible hindrance to the mining operations of the rock excavator.

It is our belief that this type of structure will lend itself to many forms of adaptation and yet be the most economical form of construction that has been investigated to date. We have carefully investigated all phases of fabrication and erection of this type housing and are firmly convinced that the cost figures we have given will prevail. Variations over the basic design in strength required and additional floor loadings for a second story will increase the price quoted per square foot, but this is to be expected. Prices for special adaptations of partitioning, including doors and hardware, will also increase this basic estimate, but this would also be true of any other type of structure.

DISCUSSION

MR. LANDENBERGER: I would like to ask if you have ever given consideration to using adaptable space frames that you could use for flat or rounded surfaces.

MR. LANDWAY: Yes, we did. We thought of several methods. Unfortunately, in order to get the maximum habitable area in a building underground for the least amount of money, it has to be borne in mind that erection is a very serious problem and a very costly one because you are working from the inside. In other words, you cannot get outside to do any structural work whatever. Now, I couldn't hear you too well; but does that answer your question? We did give thought to it.

MR. LANDENBERGER: All right.

MR. JAMES POWER (Dignum Associates, Coral Gables, Florida): In line with that erection problem, I wonder if any consideration has been given to air-supported structures.

MR. LANDWAY: We did, yes. However, our use of any type of structure was based on the fact that we have two large fabricating plants, and we wanted to design something that we could produce as well as our competitors.

FIGURES

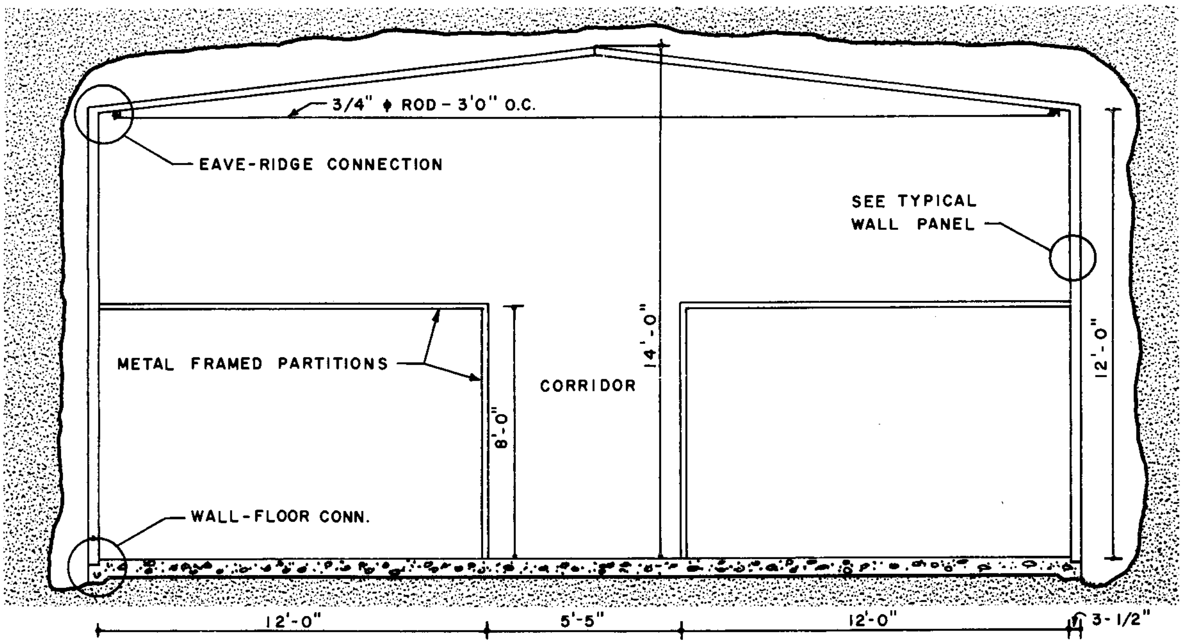


Fig. 1 — Typical single story cross section

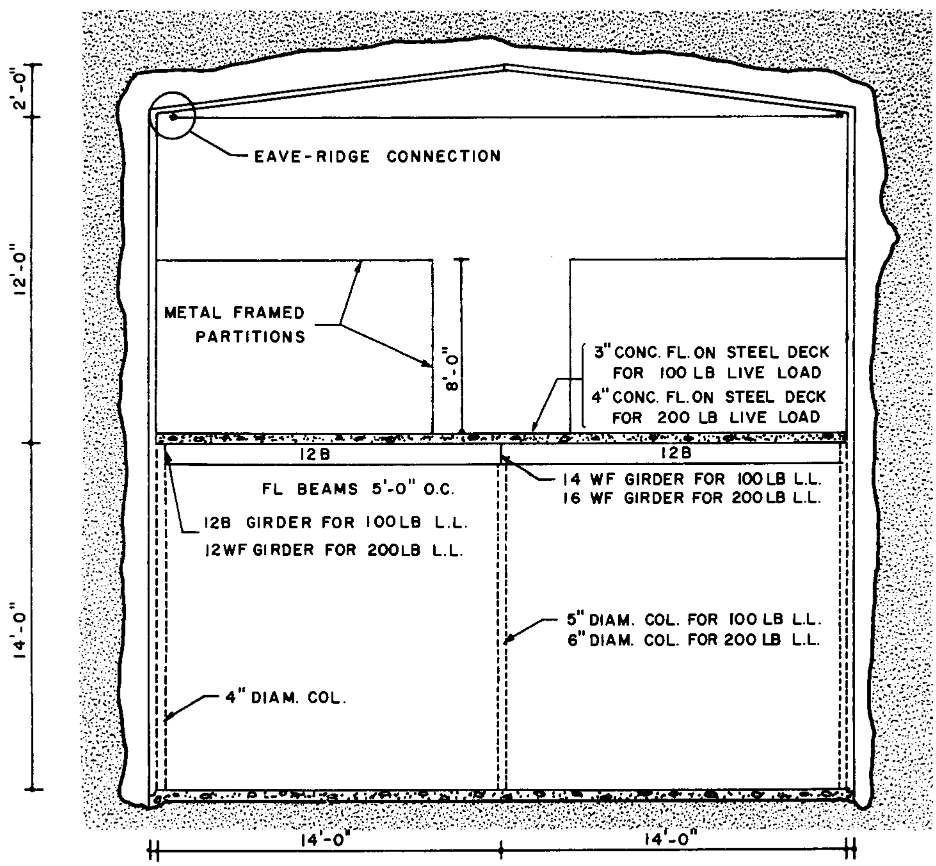


Fig. 2 — Typical cross section, two story building

Fig. 3

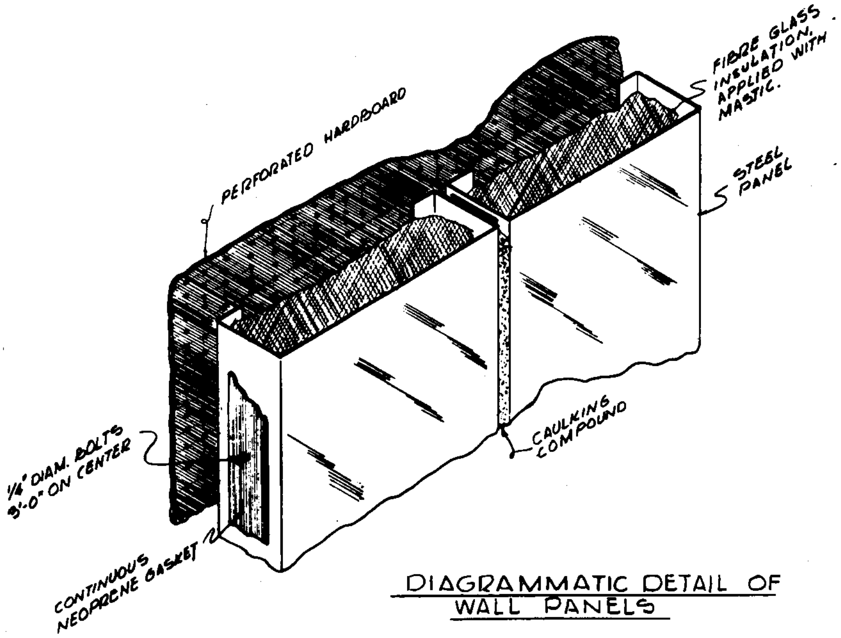

FIBRE GLASS INSULATION, APPLIED WITH MASTIC.
PERFORATED HARDBOARD
STEEL PANEL
CAULKING COMPOUND
1/4" DIAM. BOLTS 3'-0" ON CENTER
CONTINUOUS NEOPRENE GASKET
DIAGRAMMATIC DETAIL OF WALL PANELS

Fig. 4

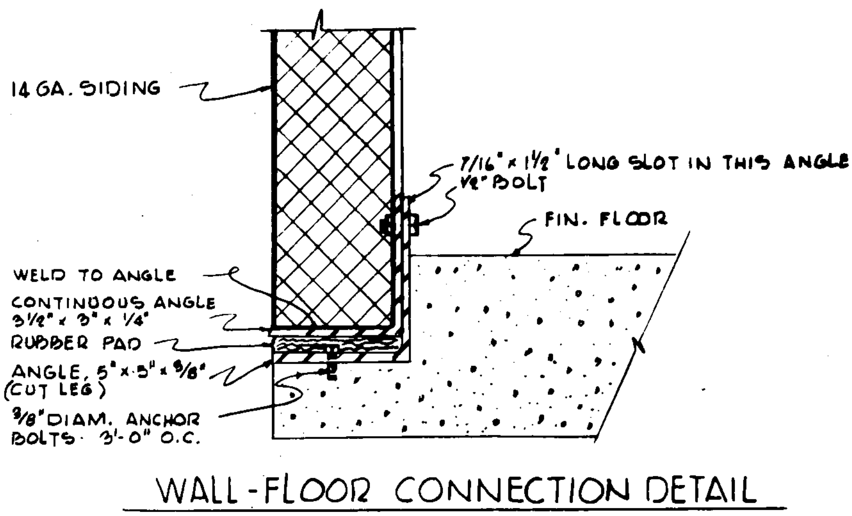

14 GA. SIDING
7/16" x 1½" LONG SLOT IN THIS ANGLE
½" BOLT
FIN. FLOOR
WELD TO ANGLE
CONTINUOUS ANGLE 3½" x 3" x ¼"
RUBBER PAD
ANGLE, 5" x 5" x 3/8" (CUT LEG)
3/8" DIAM. ANCHOR BOLTS 3'-0" O.C.
WALL-FLOOR CONNECTION DETAIL

Fig. 5

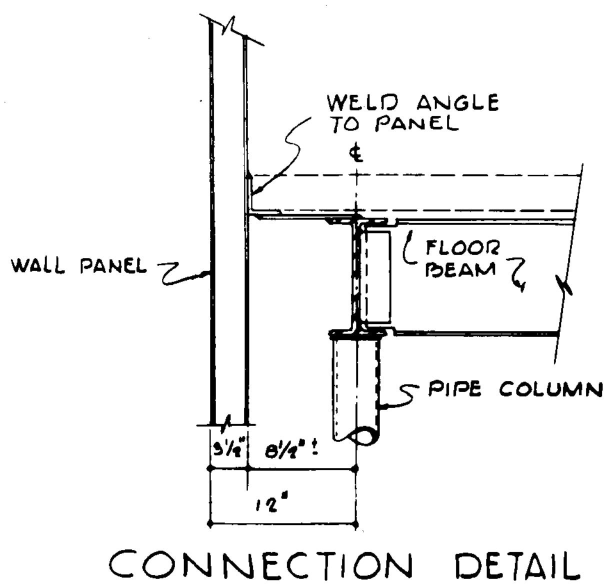

WELD ANGLE TO PANEL
WALL PANEL
FLOOR BEAM
PIPE COLUMN
3½"
8½" ±
12"
CONNECTION DETAIL

Fig. 6

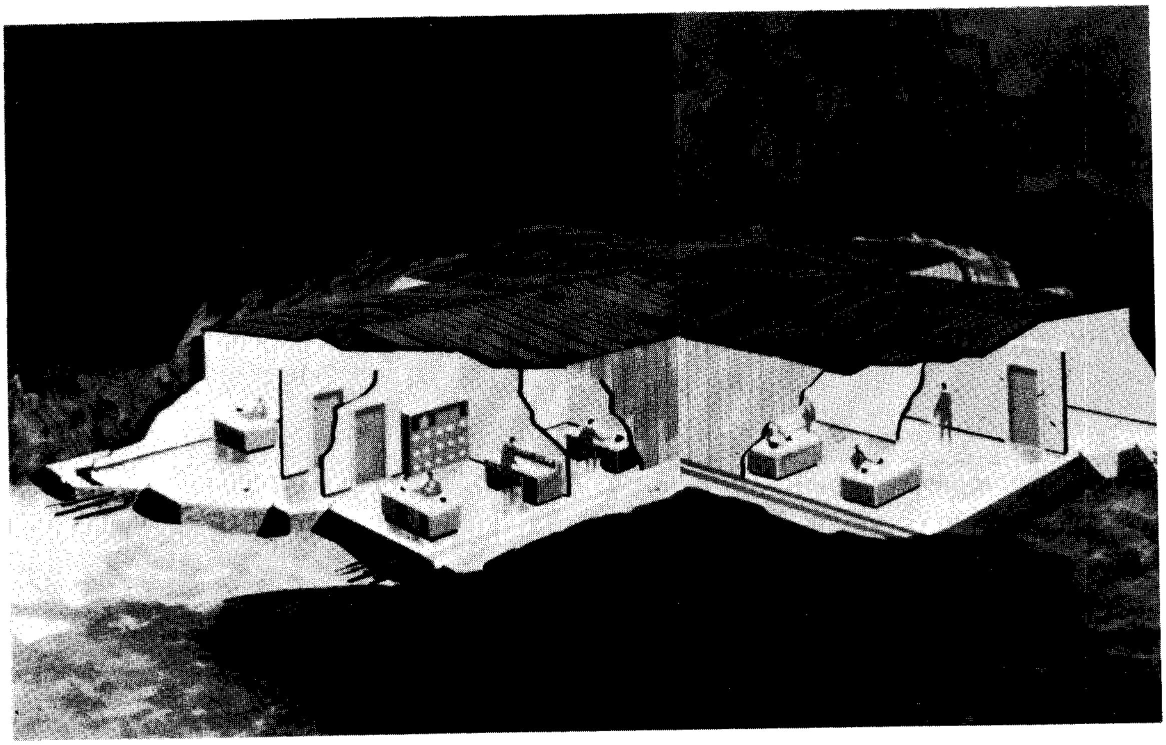


Fig. 7 — Underground shelter system
Single story

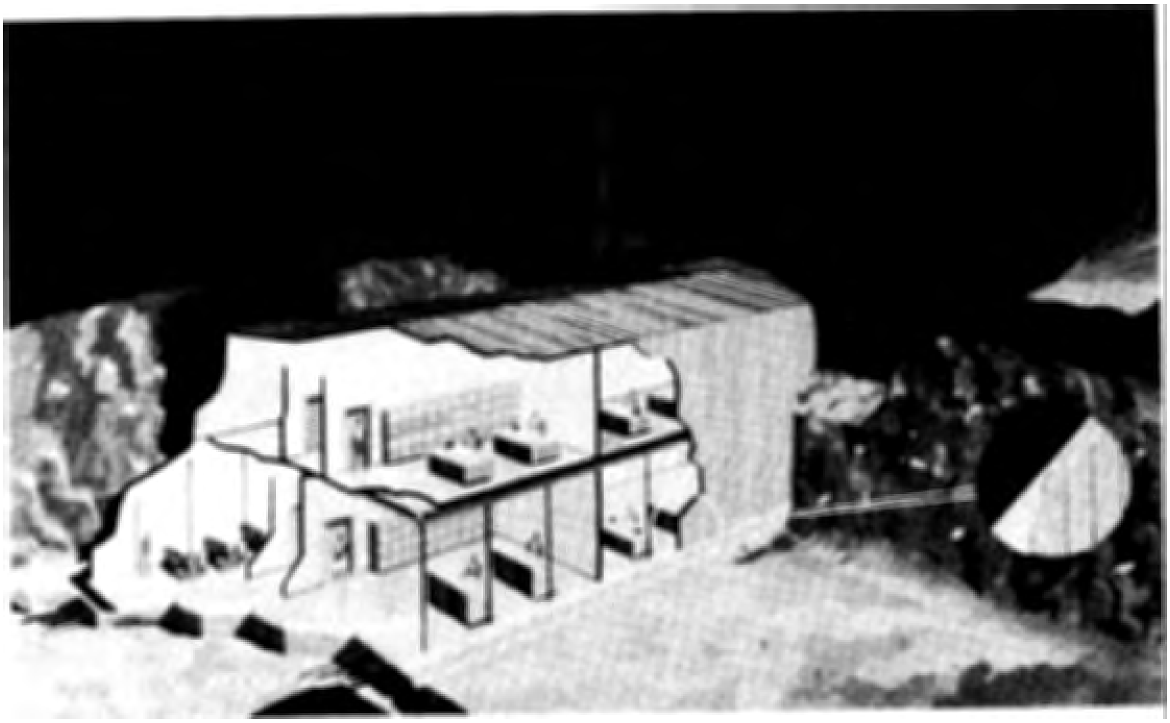


Fig. 8 — Underground shelter system
Multi-story system

PLANNING AND CONSTRUCTION OF UNDERGROUND FACILITIES FOR KEMANO-KITIMAT PROJECT

James W. Libby
International Engineering Company, Inc.

INTRODUCTION

Five hundred miles north of Vancouver, British Columbia, Canada, lies a rugged and beautiful wilderness area known as Tweedsmuir Park. Connecting the western edge of this elevated park area to the Pacific Ocean are a number of glacial fiords. Two of these fiords terminate at the mouths of the Kitimat and Kemano rivers.

The municipality of Kitimat and a large aluminum smelter plant of the Aluminum Company of Canada are located on the outwash plains of the Kitimat river. Fifty miles southeast of Kitimat, on the Kemano river, is the largest underground power plant in the world located in one of the largest, single, man-made underground caverns ever used as a permanent facility. Physical orientation of the project is shown on Fig. 1.

A description of the many problems encountered in the planning and construction of the underground facilities for the Kemano-Kitimat Project, and the engineering skill and construction ingenuity utilized in solving the problems, would require many hours to relate and would fill several volumes.

It will not be possible in the time allotted, therefore, to present more than a sample of the problems encountered. My presentation is limited to the powerhouse excavation and related features, which are probably of the greatest interest to members of this symposium.

GENERAL DESCRIPTION

As graphically represented on Fig. 2, water for power production is taken from Lake Tahtsa through an intake structure, located about 60 ft below the normal lake surface, and into a horseshoe-shaped tunnel. The water flows through the nominal 25-ft diameter tunnel for 10 miles at a slight grade to a bifurcation structure at the upstream end of the penstocks. From the structure, the water drops 2600 ft through two 11-ft diameter, steel-lined penstocks to the centerline elevation of the 140,000 HP, vertical Pelton-type turbine units. The penstocks discharge into manifolds which feed the spiral cases through eight individual 51-in. spherical valves, located in a valve chamber upstream from the powerhouse. After passing through the spiral cases, nozzles, and turbines, the water flows through individual unit tailrace tunnels to a main tailrace tunnel and channel which discharges into the Kemano river.

The location and arrangement of the tunnels, penstocks, and powerhouse were dictated by geography and physical conformity. Convenience of inter-related penstock and powerhouse access, economy in lengths of penstock and tunnels, and general geologic conditions were other significant factors. The powerhouse, penstocks, and much of the power tunnel were constructed in a massive granodiorite intrusion of the Coast Range batholith, which is gradually replaced by a metamorphic complex of greenstone, schist, and quartzite near Tahtsa Lake.

POWERHOUSE TUNNEL LAYOUT

Part of the carefully-planned pattern of tunnels and drifts is illustrated in Fig. 3. The tunnels and a portion of the main chamber shown represent approximately one-half the area of the ultimate development.

The upper left of Fig. 3 shows the penstocks approaching the manifolds and valve chambers. Both of these latter areas are hidden behind the powerhouse excavation. Curving around the left end of the powerhouse is the drainage gallery from the valve chamber. The gallery is designed to bypass water around the powerhouse in the remote event of a unit penstock or control valve failure.

The eight individual unit tailrace tunnels and main tailrace are shown downstream from the powerhouse. From right to left, projecting from the downstream face of the powerhouse, are the access tunnel, high voltage cable tunnel, and exploration tunnel which was later utilized as a ventilation tunnel.

The sequence of driving individual tunnels and drifts for powerhouse excavation was carefully coordinated to provide the most economical and satisfactory location for tunnels, penstocks, and powerhouse. First, an exploration tunnel was driven to determine rock characteristics of the proposed powerhouse site. From this tunnel diamond drill holes were drilled as the heading advanced. The holes were driven in radial patterns spaced to augment surface exploration drilling data. The pattern of joints and seams revealed by these explorations was carefully examined; as a result, the proposed site was moved further into the mountain to avoid a major shear zone, and the powerhouse alignment was altered for an improved angle of intersection with the major joints.

In connection with the exploration program, a test sphere was constructed in the rock. The sphere was 10 ft in diameter and provided with steel lining solidly backfilled with concrete. After installing SR-4 strain gages, the sphere was subjected to internal water pressures to 3600 psi maximum. Some purposes of the testing were to observe the reaction of steel lining under design conditions and to derive a value of modulus of elasticity of the rock. Measurements of deformation under load verified the design assumptions. After the penstock was completed, additional strain gages were installed and prototype testing was conducted. Results of these prototype tests were in close agreement with test sphere results. Strain gages, placed in radial holes drilled from the test sphere to solid rock, produced erratic results of only relative design value.

After the location and alignment of the powerhouse were set, the 27-ft diameter arch sections of the tailrace tunnel and the access tunnel were driven. Where the tailrace tunnel intersected the centerline of the powerhouse, 27-ft drifts were driven to the right and left for the full length of the initial eight-unit powerhouse, which is approximately 700 ft long. In addition, a drift from the exploration tunnel was driven 1100 ft along the full length of the ultimate powerhouse at about the elevation of the arch springline.

POWERHOUSE ARCH SECTION EXCAVATION

At 120-ft centers along the 10 by 12-ft exploration drift, secondary drifts were excavated at right angles to the exploration drift and out to one foot beyond the neat line of the arch section. After these drifts were completed, the full arch area was drilled and shot, forming slots which divided the arch section of the powerhouse into six segments. Seven muck raises, 8 ft square, were drilled, connecting the drift in the arch section to the lower drift.

When the first arch segment was isolated, the segment was drilled by the "long-hole" method with 55-ft horizontal holes in a pattern of concentric circles, the center aligned to the exploration tunnel. The individual circles in the pattern were spaced about 5 ft apart and all holes in the pattern filled with explosives at the same time.

With the arch segment drilled and loaded, the central circle only was fired, with the swell completely filling the exploration tunnel. As excavation continued, the exploration drift was enlarged so that two or three of the circles could be shot at once. The last shots in each section were carefully-controlled trim hole shots along the arch line. The careful control resulted in the least possible blasting shock to the remaining rock. Very little over-excavation was experienced with this method of controlled blasting.

In one section of the powerhouse, successive blasts from the exploratory drift upward disclosed weak, shattered, and blocky rock. Here the center 20 ft was carried up to the roof line and the rock heavily bolted and gunited. Progressive widening of this opening, using light shots and roof bolting, developed the full section safely without support.

No temporary structural support of the rock in the arch roof was required, but 2200 temporary "fish tail" rock bolts and 11,000 cubic feet of gunite were used to assure safety of personnel until the arch concrete was in place. The parabolic arch section, with a rise of about 37 ft and a span of 103 ft, was apparently a very satisfactory design for the type of rock.

POWERHOUSE BENCH SECTION EXCAVATION

After a couple of the arch segments had been cleaned out, and rock bolting and guniting completed, the arch section from the springline up was poured and excavation of the lower powerhouse area was started. Here again the "long-hole" method was used, with drilling commencing at the springline and extending down 75 ft to the bottom floor of the powerhouse. Approximately one-half the full width was shot at a time and mucking was performed at the lower level by shovel and trucks.

Figure 4 shows the bench cut with the arch section pour completed overhead. Notice that the excavated section from one central shot is about one-half the width of the powerhouse and extends full depth. The walls on the right and left were selectively drilled and shot out to the neat lines, where the controlled blasting effectively minimized disturbance in the rock mass.

The powerhouse cavern is 140 ft high from the bottom of the turbine pits to the crown of the arch and and 82 ft between side-walls.

DESIGN AND CONSTRUCTION NOTES

Figure 5 shows the excavation for the powerhouse completed and forms for the walls and galleries rising into position. Note the regularity of the powerhouse walls after the selective shooting.

The parabolic concrete arch was designed to thrust roof loads into the rock mass so as to reduce local high stresses in the side-walls. Roof loading above the arch is carried by the rock walls of the chamber. The concrete arch was designed to withstand only those pressures resulting from grouting between the arch and rock. The grout fill was designed to prevent initial movement of local rock areas and possible subsequent movement of a larger rock mass which might be induced by machinery vibration. The columns and beams of poured concrete are used solely to support internal loads and carry no arch loading. The vertical columns carry the 450-ton traveling crane and were tied to the rock walls for lateral support by long-stressed steel dowels.

Rock in the powerhouse area contains many dikes and extensive jointing. Four faults and resulting fracture patterns in the area induced apparent weakness in the east side of the arch and west vertical walls.

Figure 6 shows a fall-out of part of the west wall section, which had stood for a considerable period of time with no apparent distress.

Luckily, the oiler on the shovel was on the other side of the powerhouse at the time. The operator was still in his seat when the fall occurred and although the 1-1/2 cubic yard capacity shovel was completely demolished, the operator escaped with minor bruises.

Experience with major falls of this nature led to the decision to utilize long, 30- to 40-ft by 1-1/2 in. diameter bolts to permanently tie the exposed face into the rock mass, and to provide structural stability in the wall sections. These bolts, over 1500 in number, were placed in inclined diamond drill holes containing fluid sand-cement grout, the fish-tail wedges driven tight and then stressed.

Figure 7 shows a fall-out of an area on the east side of the powerhouse, at an intersection with a unit penstock tunnel that was known to be weak and which had been extensively reinforced by rock bolting and guniting. The rock bolts were from 8 to 16 ft in length and well-embedded, indicated by the fact that they remained in place after the fall. The extent and weight of the fall was so great, however, that the heads of the bolts were sheared off.

Figure 8 shows the completed powerhouse structure, revealing none of the problems encountered but standing as a monument to their solution.

Located a quarter of a mile inside the mountain, the powerhouse development required excavation of almost 220,000 cubic yards of rock, utilizing 180,000 lineal feet of diamond drilling and 127,000 pounds of dynamite.

Drilling and blasting alone required 98,000 man hours, or about 0.45 man hours per cubic yard of direct labor. In addition, scaling and temporary rock-bolting alone absorbed 21,000 man hours.

PERSONNEL

By careful supervision and extensive safety programs, the main powerhouse excavation was completed with no loss of life and a minimum of lost-time accidents. This was remarkable, considering the complexity of the operation and number of new men who had to be trained (due to the isolation of the site and severe weather conditions, personnel turnover was rapid).

CONCLUSION

Underground construction is a difficult task, but one which can be successfully accomplished if the proper teamwork between designer and construction personnel is developed. The designer must provide flexibility in his design to meet problems that arise during construction. At the present state-of-the-art of underground structure design, predictions of future events are not always reliable; consequently, a designer must be ready to completely change his thinking and designs, not tomorrow or next week, but within a few hours, to meet construction expediency, or stand the penalty of costly delays and accidents.

BIBLIOGRAPHY

1. "World's Biggest Aluminum Plant," Engineering News-Record, Nov. 29, 1951.

2. H. R. Rice, "Alcan's Nechako-Kemano-Kitimat Project," Canadian Mining Journal, June 1952 and July 1952.

3. F. H. Stephens, "Kemano," Western Miner, June 1952.

4. L. L. Wise, "Setting Up Shop in the Wilderness for Huge Aluminum Hydro Project," Engineering News-Record, Sept. 25, 1952.

5. F. L. Lawton and J. S. Kendrick, "Nechako-Kemano-Kitimat Hydro-Electric Power Development and Aluminum Reduction Plant," Engineering Journal, Sept. 1952.

6. "Kenny Rock-Fill Dam . . . a Construction Triumph in the Wilderness," Construction Methods and Equipment, Oct. 1952.

7. L. L. Wise, "World's Largest Underground Power Plant," Engineering News-Record, Nov. 13, 1952.

8. W. G. Huber, "Alcan--British Columbia Power Project Under Construction," Civil Engineering, Nov. 1952.

9. "Alcan in British Columbia . . . Construction Battles Difficult Access," Construction Methods and Equipment, Nov. 1952.

10. "Alcan in British Columbia . . . Methods Spur Underground Powerhouse, Tunnels," Construction Methods and Equipment, Dec. 1952.

11. F. L. Lawton, "Nechako-Kitimat Water Project in British Columbia," Water and Water Engineer, Jan. 1953.

12. W. G. Huber, "Tunnels and Underground Penstocks Require a Million Cubic Yards Rock Excavation," Civil Engineering, Feb. 1953.

13. "Alcan Nechako-Kemano-Kitimat Development," Engineering Journal, April 1953 and Nov. 1954.

14. Lawrence G. Ecroyd, "Progress Report on the Alcan Project," Western Business and Industry, April 1953.

15. "At Kemano, Alcan Comes Up with Another 'Biggest,'" Western Construction, April 1953.

16. F. L. Lawton, "The Kitimat Hydro-Electric Power Development Scheme," Engineers Digest, April 1953.

17. "The Nechako-Kemano Hydro-Electric Scheme in British Columbia," The Engineer, May 29, 1953, and June 5, 1953.

18. W. G. Huber, "Complex Excavation Pattern for Alcan Underground Powerhouse, Civil Engineering, June 1953.

19. "The Alcan Project," five articles from Western Construction, Nov. 1953 to March 1954.

20. "High-Head Underground Hydro Plant," Power, Feb. 1954.

21. "Kitimat," Water Power, March 1954 and April 1954.

22. "Kemano-Kitimat Hydro-Electric Scheme," Engineering, April 2, 1954.

FIGURES

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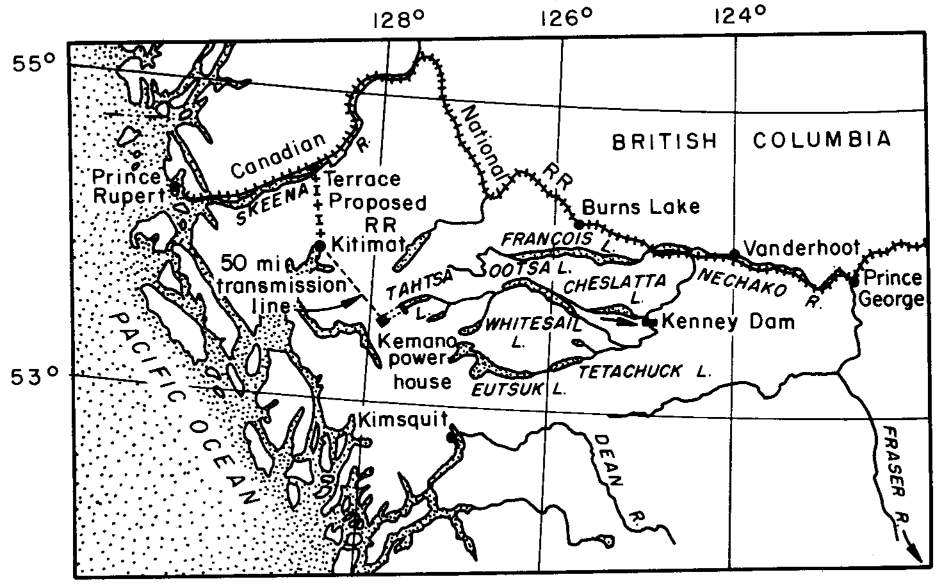

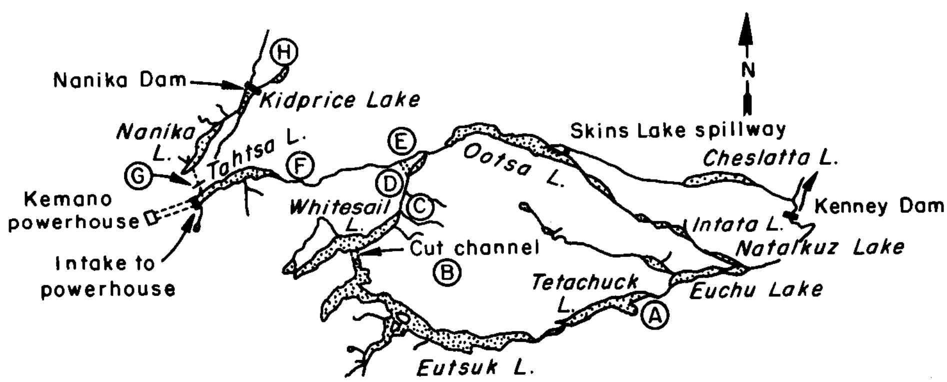

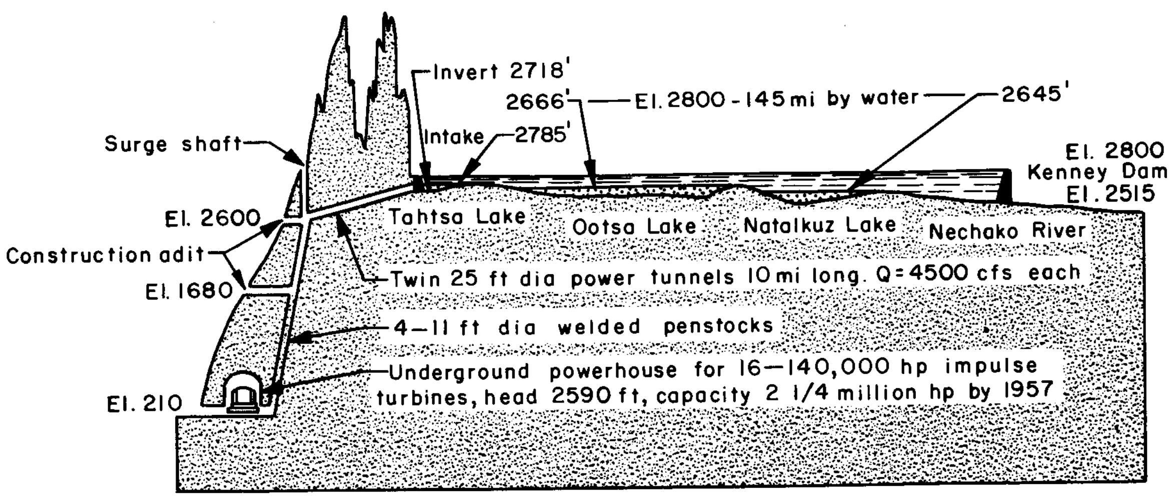

Fig. 1

Fig. 2

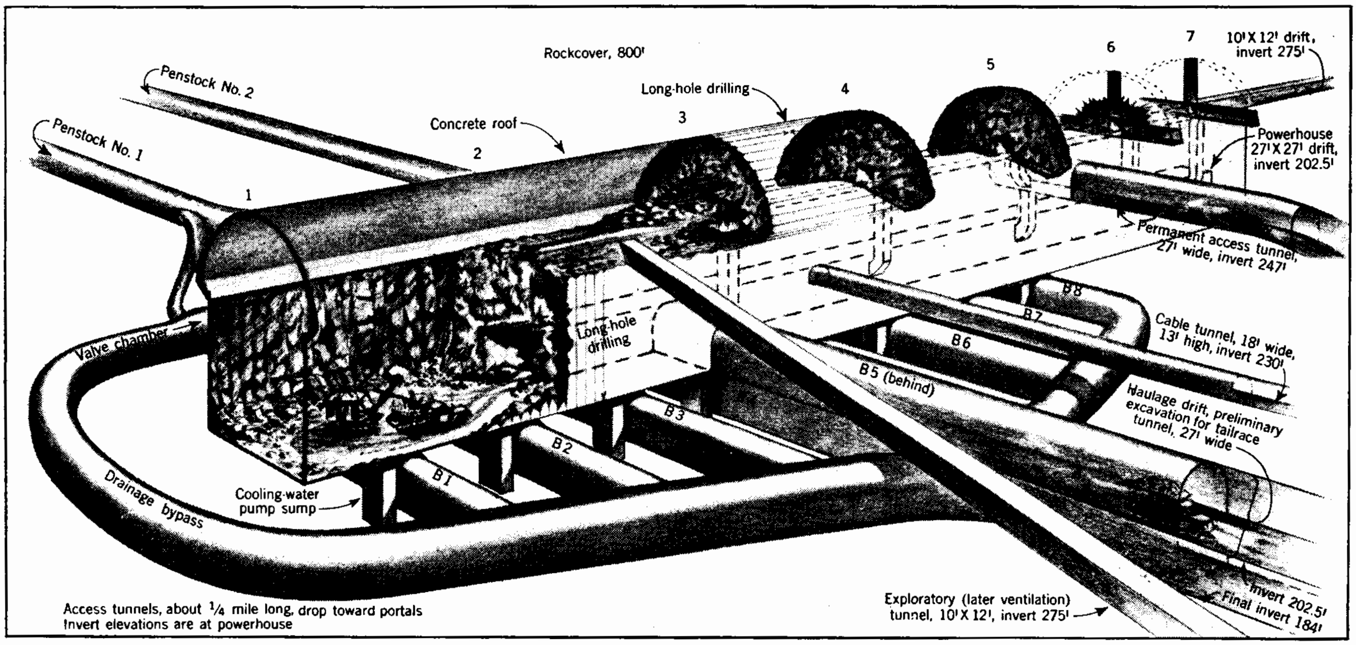
Rockcover, 800'
Penstock No. 2
Penstock No. 1
Long-hole drilling
Concrete roof
1
2
3
4
5
6
7
10' X 12' drift, invert 275'
Powerhouse 27' X 27' drift, invert 202.5'
Permanent access tunnel, 27' wide, invert 247'
Valve chamber
Long-hole drilling
B8
B7
B6
B5 (behind)
B3
B2
B1
Cable tunnel, 18' wide, 13' high, invert 230'
Haulage drift, preliminary excavation for tailrace tunnel, 27' wide
Cooling-water pump sump
Drainage bypass
Access tunnels, about 1/4 mile long, drop toward portals
Invert elevations are at powerhouse
Exploratory (later ventilation) tunnel, 10' X 12', invert 275'
Invert 202.5'
Final invert 184'

Fig. 3

Fig. 4

Fig. 5

Fig. 6

Fig. 7

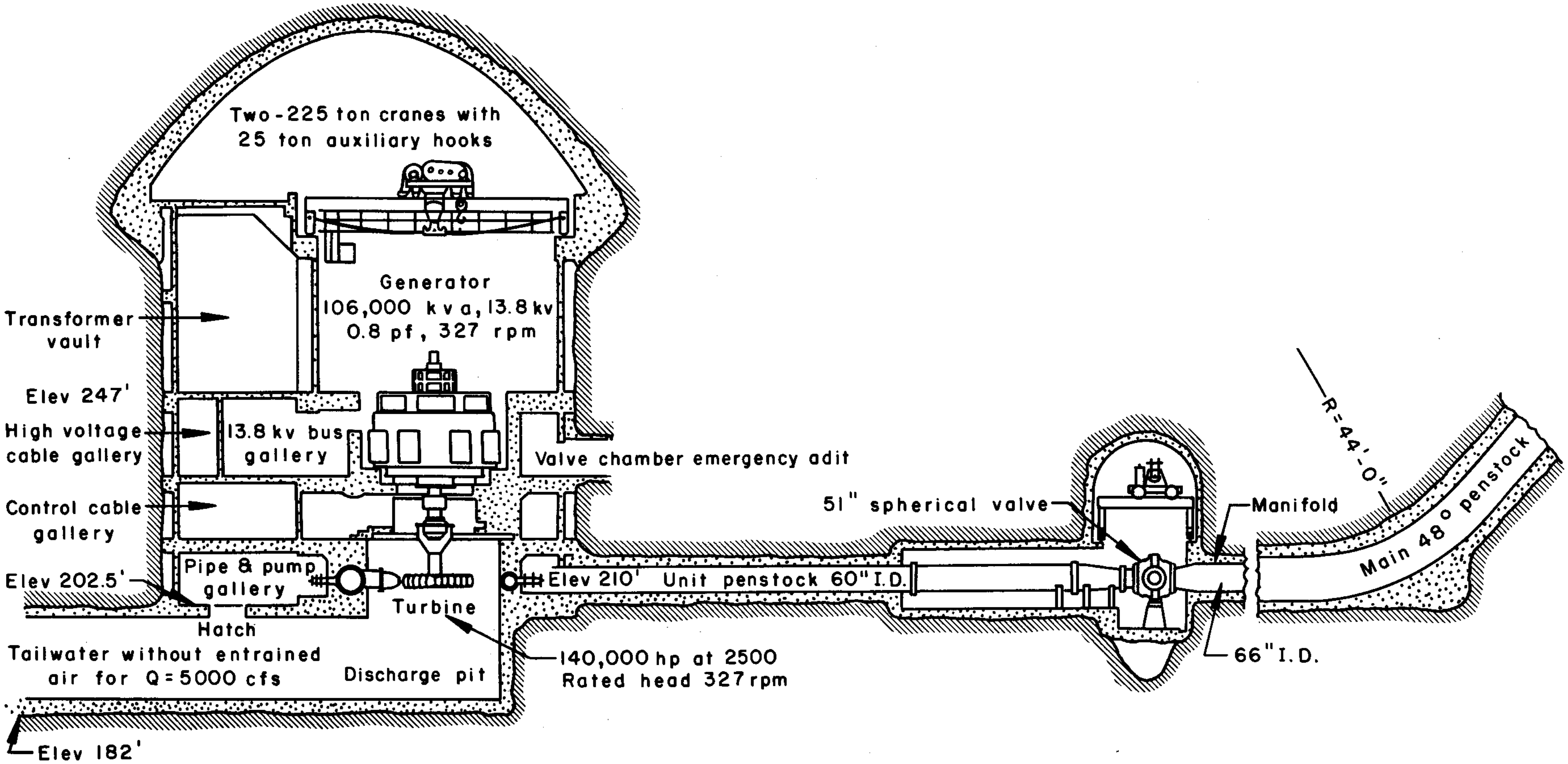

Fig. 8

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727

UNDERGROUND FEATURES OF THE HANABANILLA AND BINGA PROJECTS

John Lowe, III
Associate Partner
Tippetts-Abbett-McCarthy-Stratton

PURPOSE AND SCOPE

This paper describes the engineering and construction aspects of the underground features of the Hanabanilla Hydroelectric Project in Cuba and the Binga Hydroelectric Project in the Philippine Islands. The underground power generating facilities of these projects involve large caverns and intricate passageways deep in the earth. Problems involved in the design and construction of such facilities are similar in many ways to problems encountered in the design and construction of underground protected facilities. The discussion presented herein is limited to those aspects of the design and construction of the caverns, tunnels and their appurtenances which would also apply to underground protected facilities. Included are such items as access, ventilation, waterproofing, heavy machinery handling and sanitary facilities.

Because the design problems for the Binga Project are similar in many instances to those of the Hanabanilla Project, a detailed discussion of the Hanabanilla Project is presented first and is followed by a discussion of only those items of the Binga Project which are not similar.

HANABANILLA PROJECT

Project and Location

The Hanabanilla Hydroelectric Project is located in the Trinidad-San Juan Mountains of south-central Cuba, approximately 31 miles from the city of Cienfuegos. The project will provide a source of low cost peaking electricity for Cuba's rapidly expanding industrial and domestic needs by impounding the waters of three major rivers in the project area: The Hanabanilla, the Negro and the Guanayara. The project consists of two rock and earthfill dams which create a reservoir of 76 billion gallons capacity, and an underground power station of 27,000 kw capacity, together with waterways, spillway dikes and necessary appurtenant facilities. Total cost is estimated to be $15 million.

Underground Features

The underground features of the project consist of a vertical penstock shaft; an underground power station; an underground valve chamber; an outlet or tailrace tunnel and an inclined service tunnel. The location of these facilities, as well as Hanabanilla Dam, is shown in Fig. 1. The other dam, named the Jibacoa Dam, is located at the other end of the reservoir. An isometric view of the underground facilities is shown in Fig. 2.

The geology of the project site was studied by airphoto interpretation and field investigations. Deep NX borings were made to explore the penstock, service tunnel, tailrace tunnel and power station sites; four of these borings were drilled in known fault zones along the tailrace tunnel alignment. The locations of these borings are shown in Fig. 8. These explorations indicated that the formations in the areas to be excavated were predominantly dioritic gneiss, a granitoid-layered type of rock, with some cleavage. Joints are spaced and oriented at random. The excavation of the caverns and tunnels proved this rock to be of excellent quality.

The Penstock

The penstock consists of a 13-ft diameter, 841-ft long vertical shaft (Fig. 3). The shaft was excavated from the top downward by the bench method. There were two advantages in sinking the penstock shaft by the bench method of drilling and blasting. First, hand mucking was made easier because the blast usually threw most of the muck to one side of the shaft and thus facilitated hand shoveling the blasted material into the 2-3/4 cu yd bucket in the cut. Two buckets were utilized, one being loaded while the other was being hoisted for dumping into the spoil chute. Second, this method provided a sump to collect the water that drained into the shaft, and made possible a dry bench for drilling and blasting operations.

The diorite gneiss encountered in the shaft was of excellent quality. The explosive loading factor averaged 4.0 lb of dynamite per cu yd of solid material. The quantity of material excavated was 4000 cu yds.

Even though the penstock shaft was in excellent rock, it was necessary to fully support it with steel ribs, spaced 4 ft center to center, to protect the workmen from rock falls. The penstock is to be lined with steel plate and Prepakt concrete to withstand internal and external pressures. The design is based on a maximum internal pressure of 485 lb/sq in., two-thirds of which is resisted by the rock and one-third by the steel liner. The steel and concrete lining is adequate to take the maximum external pressure of 485 lb/sq in. Prepakt concrete is to be used to eliminate shrinkage and obviate contact grouting. The 8.65-ft steel liner varies in thickness from 3/8 in. at the surface to 1-9/16 in. at the bottom of the shaft. Between the liner and the rock wall is a minimum thickness of 2.25 ft of "Intrusion Prepakt" concrete which is to be made with aggregate having a maximum size of 3 in. High pressure grout holes will be drilled through existing grout outlets in the steel liner into the concrete and rock. Grouting pressures are expected to be limited to 150 lb/sq in., with the field engineer determining the pressure used.

The Service Tunnel

The service tunnel is an 1115-ft shaft inclined at 42° to the horizontal. This shaft extends from El 1084 ft at the access house on the north abutment of Hanabanilla Dam to El 336 ft at the central power station (Fig. 4). An inclined shaft was chosen in preference to a vertical shaft for two general reasons: to expedite construction and to facilitate movement of heavy equipment into and out of the power station and valve chamber.

In general, the use of an inclined shaft will expedite construction because (1) it is easier for personnel to work in a sloped shaft than in a vertical shaft and (2) by inclining the shaft at an angle approximately equal to the angle of repose of the muck and excavating from the bottom, the material which is blasted can be made to fall to the base of the shaft and can then be removed through an access tunnel, whereas in vertical shafts which are often excavated from the top, the material must be lifted out. In actuality, this advantage was not fully realized at Hanabanilla because access to the bottom of the shaft was not initially attained in time due to unforeseen delays and consequently the shaft excavation was started from the top.

Movement of equipment is facilitated by the choice of an inclined shaft because a railway can be used, as compared to an elevator which must be used in a vertical shaft. The use of the railway is also advantageous because less hoist capacity is required to move a given load.

A specific advantage gained by the use of the inclined shaft at Hanabanilla is that a better location for the access house was obtained without the necessity of shifting the location of the power station.

The tunnel cross-section is a modified horseshoe with an unfinished width of 16 ft and a height of 18.5 ft. The tunnel was excavated from the top downward. However, in the later part of the excavation when a pilot tunnel was holed through from the power station chamber, the tunnel muck was chuted down the pilot tunnel and removed through the lift operating in the vertical shaft. When working from the top downward, mucking was done by hand into a 3 cu yd skip, riding on rails. A loading ratio of 3.5 lb of explosives per cu yd of rock was used for the 16,000 cu yds of rock excavated. The upper 180 ft reach in the zone of weathering of the diorite required steel supports (Fig. 5). Below 180 ft the diorite is of nearly massive character. In this section steel supports, and rock bolts, 3/4 in. in diameter and having a minimum length of 6.6 ft, were used where necessary.

The tunnel will have a concrete lining with a minimum thickness of 1 ft on the sides and approximately 1.8 ft at the crown (Fig. 6). Grout pipes 1-1/2 in. in diameter will be placed on 3-ft centers in the concrete lining so that after the concrete sets grout can be forced into any voids between the lining and the rock. Rubber water stops will be used between the joints which are located at 35-ft intervals.

In order to take care of the drainage, standard cast iron pipe, 2-1/2 in. in diameter and extending a minimum distance of 1 ft into the rock on the sides of the tunnel, will be set before concreting (Fig. 7). These horizontal pipes connect to vertical 2-in. cast iron pipes which in turn connect to 6-in. drains located on each side of the tunnel for its full length. Two inch drain pipes will also be drilled into the floor and connected to the 6-in. main drain. The drainage water, after reaching the power station, will be piped to a sump and then will be pumped to the tailrace tunnel.

The upper part of the tunnel will be partitioned off for an exhaust air duct (Fig. 7). A staircase and railing will be placed on one side and rails for an inclined railway with a wood safety rail will be placed in the center of the tunnel.

The Tailrace Tunnel

The Hanabanilla River drops from El 1066 ft at Hanabanilla Dam to El 246 ft, some 4 miles straight line distance downstream (Fig. 8). Such topography is well-suited to tunnel conveyance of water. The tailrace tunnel will return the water to Hanabanilla River after it leaves the generating turbines. The tunnel is 22,000 ft in length with an unlined diameter of 14.1 ft. Total excavation of 160,000 cu yds of rock is being made from the outlet portal end. Daily tunnel progress ranges from 30 to 55 ft using an explosive loading factor of 3.0 lb of dynamite per cu yd of material.

Larger faults observed on the surface have generally been found to be tight at tunnel grade. The faults encountered have steep dips and cross the tunnel at high angles; they consist of 1- to 200-ft-wide zones of sheared, slickensided and crushed rock. Several gallons of water a minute flow from most of these faults. One fault zone required high-pressure grouting to shut off inflowing water.

In reaches of blocky gneiss, steel supports are located at 4-ft spacing (Fig. 9). In 3.4 miles of completed tunnel 366 sets of supports were used. Present plans call for concreting only the supported reaches. For concreted sections, the tunnel will be horseshoe-shaped with a concrete lining having a minimum thickness of 1.6 ft at the crown and 1.0 ft at the invert.

POWER STATION AND VALVE CHAMBERS

General

A chamber to house the power generating equipment will be excavated near the base of the penstock shaft (Fig. 10). Adjacent to this chamber and at the base of the penstock shaft will be the valve chamber which will house the manifold and valves. Three branch tunnels will lead from the valve chamber to the power station chamber. At one end of the valve chamber will be the access tunnel to the power station; and at the other end will be the drainage tunnel which ends in a small chamber. The three draft-tube tunnels, the surge shaft and the tailrace tunnel also terminate in this small chamber.

Excavation

The excavation of the 19,227 cu yd of rock for the valve chamber and power station chamber is now underway. The arch of the valve chamber was excavated first (Fig. 10). The remainder of the excavation was benched after concreting the arch. First, the center section, 9 x 12.5 ft was excavated, then the side sections using the previous center section as an open cut. Mucking was done by scraper units pushing into muck raises. From the muck chute, a 3-1/2 cu yd skip was loaded, raised through the shaft and automatically dumped into a 33 cu yd muck bin from which the muck was hauled to the spoil area. After the underground chambers are completed, the muck chutes will be backfilled and grouted. Since completion of the service tunnel, the muck from the power station blasting operations has been chuted into the service tunnel skip for removal while the shaft facilities were used for concreting the valve chamber arch. The work of excavating the underground chambers is behind schedule due to delays caused by revolutionary activity during most of 1958 and other reasons.

Details of Valve Chamber

The valve chamber is 99 ft long, 29.5 ft wide and 40 ft high (Fig. 11). The roof will be supported by a parabolic reinforced-concrete arch having a minimum thickness of 1.6 ft. The excavation for the arch at the springline is wider than that for the lower part of the valve chamber by 8.5 ft. Where required, the sides will have a 1-ft reinforced-concrete lining anchored to the walls with No. 8 anchors on 10-ft centers (Fig. 12). Two-inch diameter weepholes will be placed in the floor and walls on 10-ft centers. Drainage is to a gutter running down the center of the floor.

A monorail capable of supporting a 10-ton load will be fastened to the roof with two 1-3/8 in. bolts every 10 ft (Fig. 13). Two-and-one-quarter-inch holes will be drilled 15 ft into the rock. The bolts are grooved for six inches on the ends so that when driven against a steel wedge placed in the bottom of the hole, the ends spread apart and wedge the bolt into the hole. The bolt is then grouted to obtain a tight fit in the hole.

A bulkhead has been designed to separate the access tunnel and the valve chamber (Fig. 14). The bulkhead is to be constructed by forming a 1.6-ft thick reinforced concrete wall across the access tunnel. A rectangular opening, 9 x 10 ft, will be left in the wall for a door made of 3/16 in. steel skin plate welded to 6-in. channels. The door will shut against rubber seals fastened to channels embedded in the concrete wall. A small watertight door will be made in one side of the larger door.

Details of Power Station Chamber

The power station chamber excavation is 121 ft long, 39 ft wide and 84 ft high (Fig. 15). The bottom of the excavation is complicated by the numerous cuts for the turbine foundations. The roof will be supported by a reinforced-concrete parabolic arch similar to that constructed in the valve chamber (Fig. 16). There is one major difference, however, as there will be a drip ceiling constructed below the roof arch. The drip ceiling is a reinforced-concrete arch shell, having a thickness of only 3.15 in., which is stiffened every 24 ft by ribs having a thickness of 8 in. At the crown the drip ceiling has adjustable dampers and a walkway. Water dripping on the ceiling is drained off at the sides in 3-in. cast iron pipes.

A 50-ton trolley crane will run the length of the power station. The crane will be supported by girders and columns (Fig. 17). The columns will be located 10 in. from the rock wall but will be anchored to the rock at the top by means of dowels grouted into 2-in diameter holes. The spaces between columns are filled with concrete blocks to form a curtain wall. The walls of both the generator and turbine rooms will be curtain walls. The basement walls will be formed against the rock face.

Drainage

In the power station, provision must be made for drainage of water from the service tunnel, drip ceiling, valve chamber, floors, generators, compressor, after-coolers and toilets.

Drainage from the drip ceiling flows into vertical 3-in. cast iron pipes placed between the curtain walls and rock (Fig. 18). These discharge into gutters in the basement floor; the gutters, in turn, drain into a wet sump. Seepage water from the service tunnel and valve chamber and water from the floors, generators and compressor after-coolers is also piped to the wet sump. Water from the wet sump is pumped into the tailrace tunnel through a 10-in. pipe. Water from the lavatories, toilets and showers will be drained through 4-in. pipes to a septic tank and then to a pneumatic sewage ejector which connects to the tailrace tunnel. The sewage is chlorinated before ejection into the discharge line (Fig. 19).

Ventilation

Air for ventilating the power station and valve chamber will be drawn into the access house through intake louvers at the top of the building (Fig. 20). The air moves down the service tunnel at the rate of 64,000 cu ft per minute. After passing through the power station and valve chambers, the air is exhausted through the air duct which occupies the upper part of the service tunnel. Two 32,000-cu ft per minute exhaust fans in the access house at the top of the service tunnel push the air through automatically counterbalanced louvers.

Of the total volume of air passing down the service tunnel, 27,850 cu ft flows into the generator floor and 36,150 cu ft goes to the turbine floor. From the turbine floor, 1500 cu ft flows through the access tunnel to the valve chamber and is returned to the turbine floor through an 18-in. duct located in the access tunnel. Air from the turbine room flows through grills in the curtain wall and rises to the generator floor. From the generator floor the air passes through the dampers in the drip ceiling and back to the exhaust air space in the service tunnel.

BINGA PROJECT

General Description

The Binga Hydroelectric Project is located in the north-central part of the Island of Luzon, Republic of the Philippines, approximately 9 air-line miles east of the city of Baguio. The project, situated on the Agno River, has been designed to operate in tandem with the Ambuklao Hydroelectric Project located approximately 11.8 miles upstream along the river. Ambuklao, which has an installed capacity of 80,000 kw, will free stored releases from its large reservoir to Binga as well as generate power at its own site. Binga, with a somewhat smaller reservoir, will provide peaking capacity in addition to generating prime energy.

The Binga Project consists of a rock and earthfill dam which creates a reservoir of approximately 29 billion gallons capacity, an underground power station of 100,000 kw capacity, and necessary appurtenant facilities. Total cost of the project is estimated to be $52 million.

Geology

The Agno River Basin is formed by two almost parallel mountain ranges of the Central Cordillera System. The principal rocks of the Agno Valley in the vicinity of the site are a series of metamorphosed andesites and sedimentary rocks intruded in places by diorite and other igneous rocks. The members of the metamorphosed series were laid down as lava flows, volcanic ash and water-deposited sediments. Later they were subjected to mountain building forces which produced numerous fractures and incipient fractures. The lavas and sediments were metamorphosed and their original characteristics altered. All underground work is in rock which is far from the best and requires special skills to excavate and support.

Underground Features

The underground features consist of an 18.5-ft diameter horseshoe power tunnel 2,400 ft in length; two steel-lined, inclined penstocks, 12 ft in diameter, which branch into four 8-ft diameter penstocks just upstream of the power station; a 50-ft wide, 255-ft long, 93.5-ft high power station which lies 300 ft below the surface, an electrical cable and ventilating shaft, and an equipment access shaft rising to the surface from either end of the power station; a tailrace tunnel which extends 1.3 miles from the power station to the downstream reaches of the river (Fig. 21). Total underground excavation amounts to 387,000 cu yds.

The Binga Project differs from Hanabanilla in that (1) it has a long pressure tunnel, a surge tank and inclined penstocks; (2) access to the Binga facilities is by vertical shafts as compared to inclined shafts; (3) Binga's caverns are much larger than Hanabanilla's and are excavated in rock which is more fractured than that at Hanabanilla.

Excavation

The power tunnel excavation was made from a horizontal access tunnel which intersected the power tunnel center line just upstream of the surge tank. It was then possible to excavate a pilot raise in the surge tank while advancing the main heading towards the power intake. At the same time, the surge tank excavation which proceeded from the top, the power intake open-cut excavation and portal work was in progress.

The two access shafts to the power station were excavated simultaneously to their full depth and from the cable and ventilating shaft a tunnel was excavated in the power station area to intersect penstock No. 4; the excavation then followed this penstock upstream to its intersection with penstock No. 3. Here a cross cut was made to the intersection of penstocks No. 1 and 2. From these two intersections, pilot raises were excavated in the inclined penstocks toward the surge tank and all branch penstocks were simultaneously advanced toward the power station.

Also, from the bottom of the cable and ventilating shaft, a transit tunnel was excavated around the power station to intersect the draft tubes at the bottom of the surge chamber. This permitted advancing the draft tube excavation in both directions and excavating raises to the crown of the surge chamber and up to the ground surface to form a ventilating shaft which is also a permanent feature. This approach permitted all work to continue during the lower stages of excavation in the power station.

At the same time, the crown of the power station was excavated and concreted. A raise was excavated from the bottom of the power station near penstock No. 4 to the crown area and the power station excavation was continued from the top, utilizing the raise as a loading chute with all muck being taken out at the bottom level.

Rockbolting

Originally, while connecting the two shafts of the power station through the power station roof, vertical timbering was used. The maze of timber was so obstructive to both excavation and concreting that other means had to be utilized.

The system adopted was the patented Swedish SN method of rock bolting. The SN method utilizes a special grouting machine which introduces grout mortar into the drill hole, after which the reinforcement bar is inserted.

By utilizing this method of rock bolting it was possible to remove approximately 90 per cent of the timber in the power station crown area, thereby expediting the work. This method was also used in the lower elevations of the power station where both the upstream and the downstream walls were hazardous. It was thus possible to continue the required excavation to the lowest elevations and also to make the intersecting excavations without timber or steel supports. In the wall areas the bolts, which varied from 10 to 20 ft in length, were spaced approximately 3.3 ft center to center. In most cases the bolts were placed pointing upward at about 30° to 45° from the horizontal; some intermediate 20-ft bolts were placed horizontal in order to integrate a larger mass of supporting rock.

Pull-out tests were conducted on several bolts embedded in various types of rock and in all cases the bars withstood a load in excess of 20 tons. Failure ultimately occurred in the bar itself. An embedment of 3 to 5 ft was sufficient to develop full bar strength.

Use was also made of split rock bolts, wire netting, and lacing formed by welding reinforcing bars to the ends of the rock bolts.

Rate of Excavation

No world records were made in the excavations but considering the type of rock to be worked, it was accomplished without major mishap in a remarkably short period of time. In the power station a maximum of 8000 cu yd was excavated in one month using three 8-hour shifts. The excavation, including penstocks, draft tubes and surge chambers, involved 96,000 cu yd and required 15 months.

CONCLUSIONS

Large, rather complicated underground facilities are being constructed deep underground for power generating purposes. The underground locations are chosen for reasons of over-all economy. The experience gained in the design and construction of these facilities should be of value in the design and construction of deep underground protected facilities.

ACKNOWLEDGMENTS

The owner of the Hanabanilla Hydroelectric Project is the Primera Central Hidroelectrica Cubana, an agency of the Cuban Government. Design and supervision of construction is by Tippetts-Abbett-McCarthy-Stratton, Engineers, New York. Leonard A. Lovell is Partner-in-Charge of the Project and Alvin Goodman is Project Engineer for Design. Philip S. O'Shaughnessy represents the firm in Cuba and is Resident Engineer. Russell Roddy, Jr. is Assistant Resident Engineer. Excavation of underground facilities is by Tecon Corporation of Dallas, Texas. Other site work is by the Cuban firm of Ingenieria G. del Valle, S. A.

The owner of the Binga Project is the National Power Corporation of the Philippines of which Filemon M. Zablan is Manager, Jose O. Lahoz is Chief Engineer and Crispin T. Ubaldo is Project Engineer. The firm of Tippetts-Abbett-McCarthy-Stratton, associated with the Engineering and Development Corporation of the Philippines, are consultants to the engineering staff of the National Power Corporation. For Tippetts-Abbett-McCarthy-Stratton, Leonard A. Lovell is Partner-in-Charge of the Project, Clarence Freeman is Project Engineer for Design and Henry Neve is head of the Hydraulic and Hydroelectric Department; Daniel Abramowitz is Manager for the firm in the Philippines and Uno Engstrom is Resident Consultant. For the Engineering and Development Corporation of the Philippines, Filemon C. Rodriguez is President. The contractors are Philippine Engineers' Syndicate, Inc. and Widmark and Platzer AB, Stockholm.

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The author is indebted to Carneal Smith, Soils and Foundation Engineer, of Tippetts-Abbett-McCarthy-Stratton, who gave extensive help in the preparation of this paper.

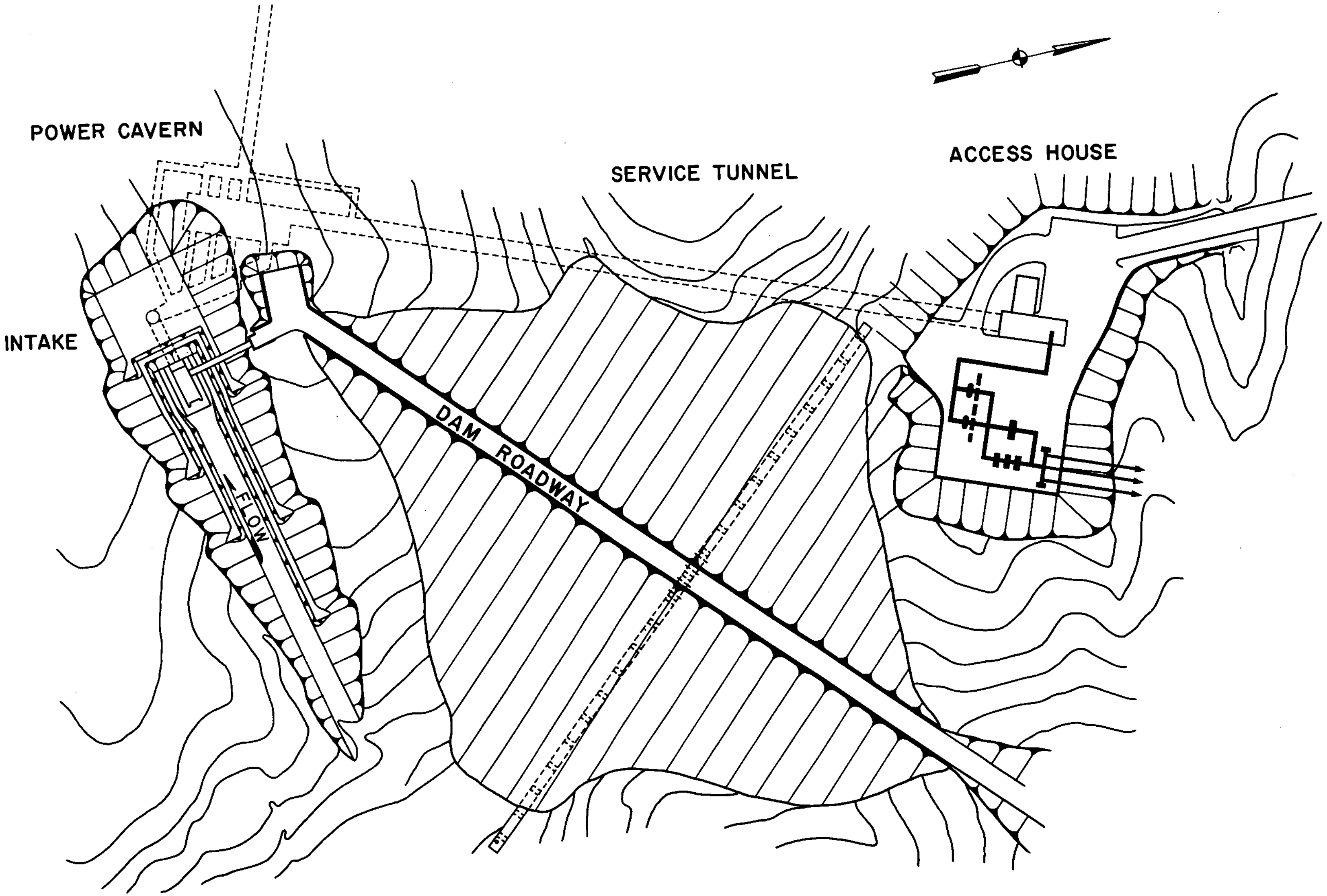

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FIG. 1 PLAN – DAM SITE – HANABANILLA HYDROELECTRIC PROJECT

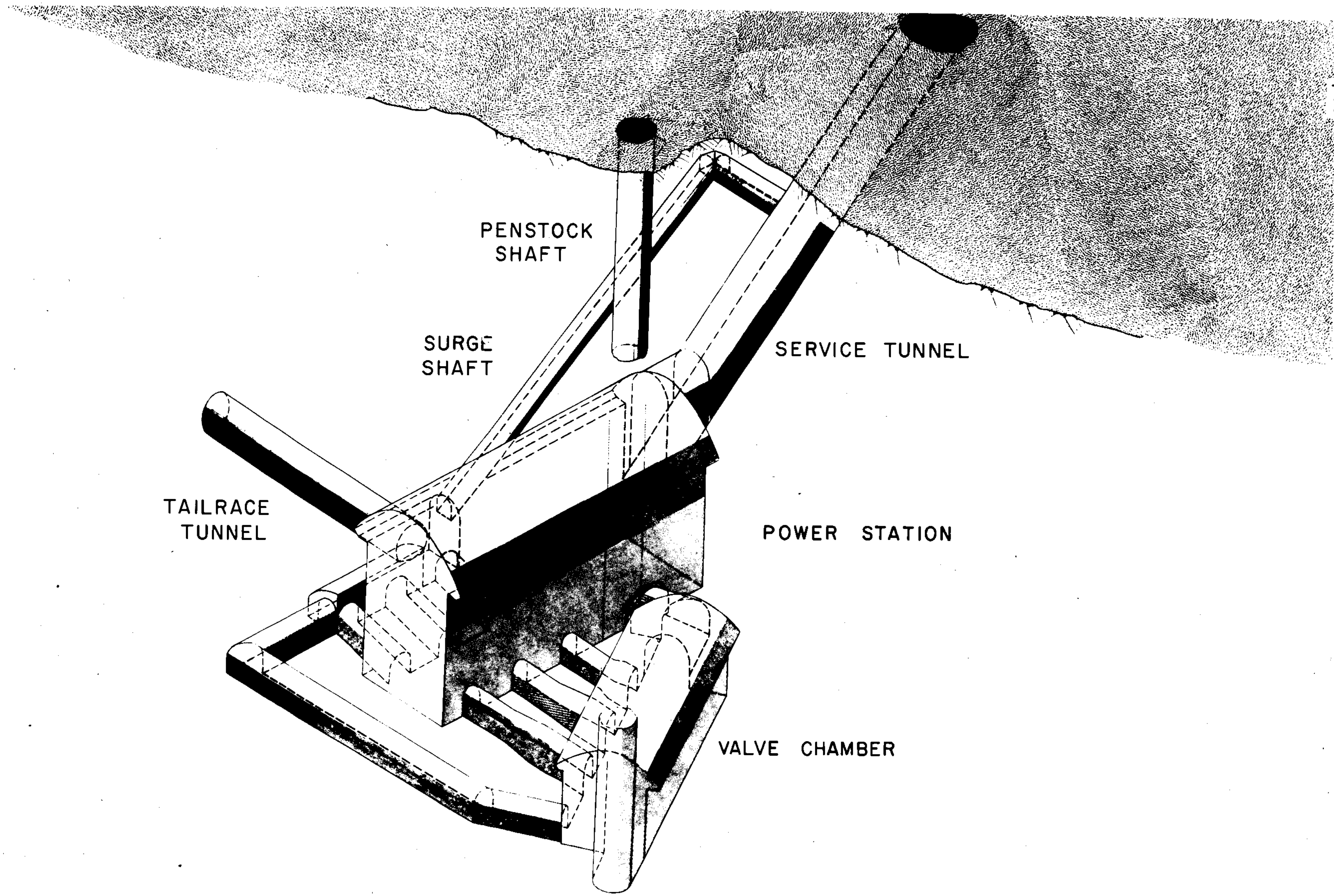

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FIG. 2 CAVERNS AND TUNNELS - HANABANILLA HYDROELECTRIC PROJECT

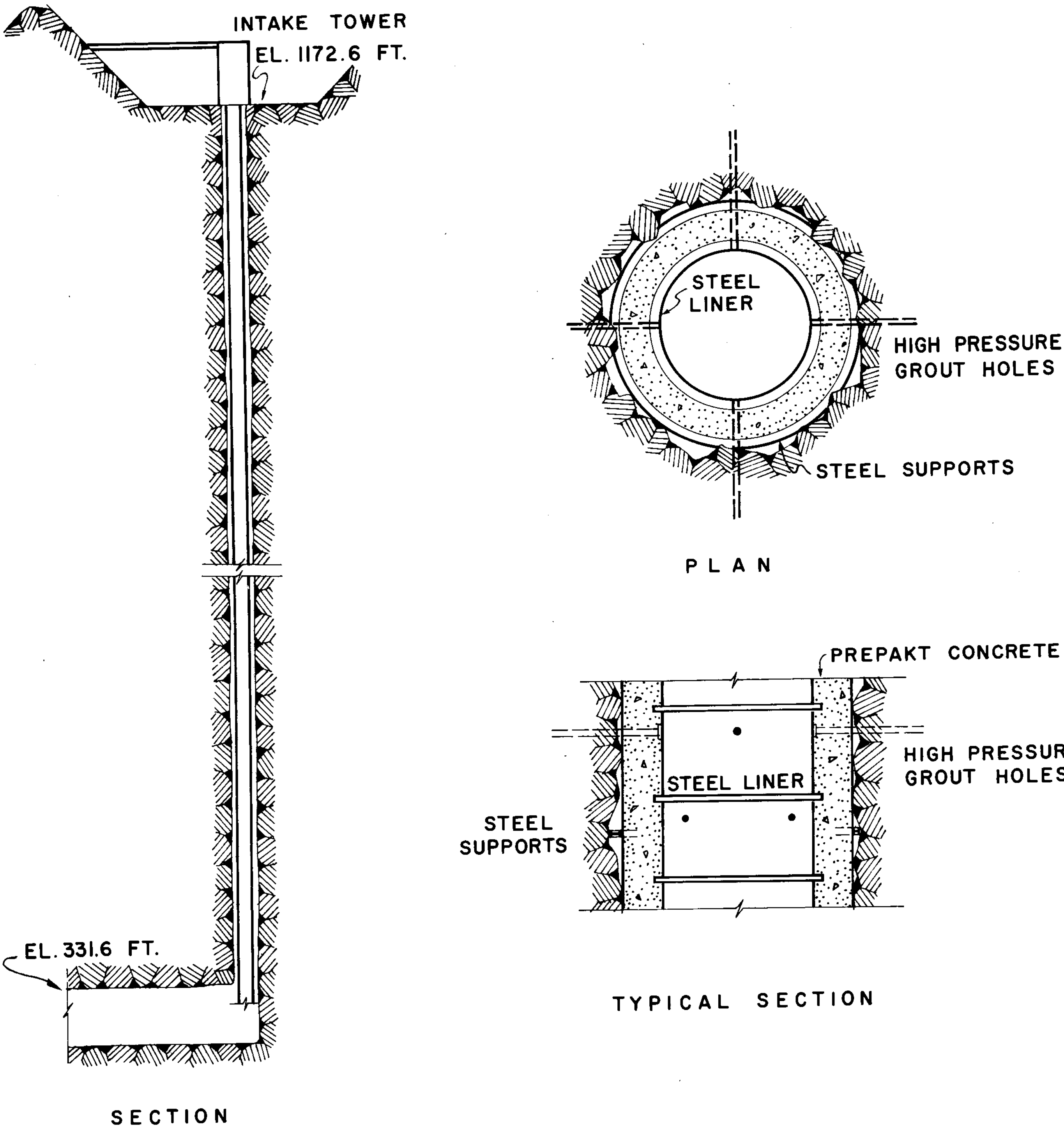

FIG. 3 PENSTOCK SHAFT - HANABANILLA HYDROELECTRIC PROJECT

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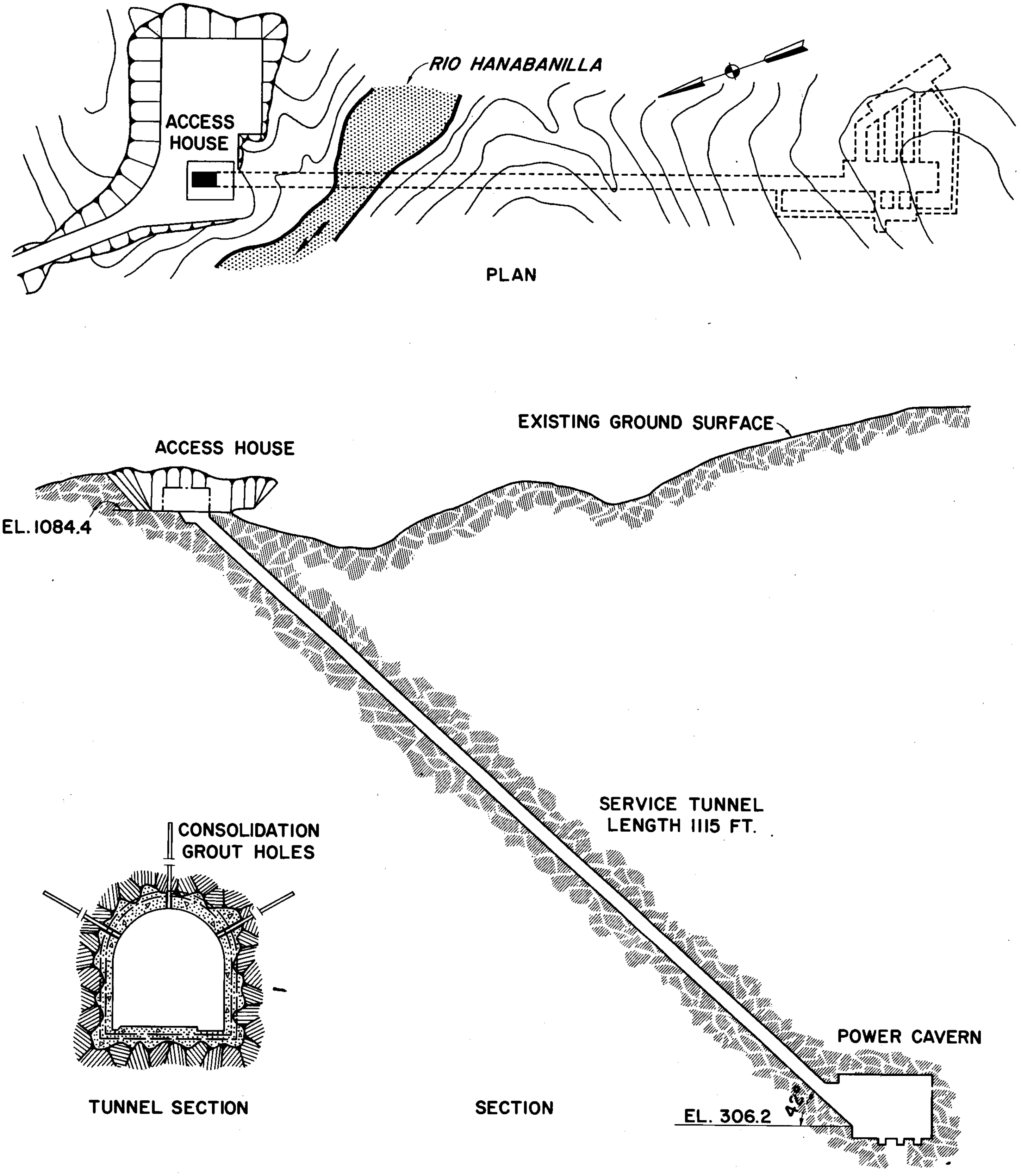
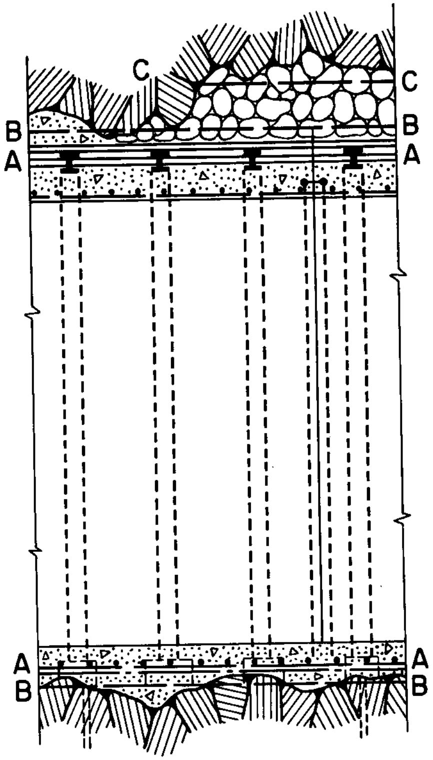


FIG. 4 SERVICE TUNNEL - HANABANILLA HYDROELECTRIC PROJECT



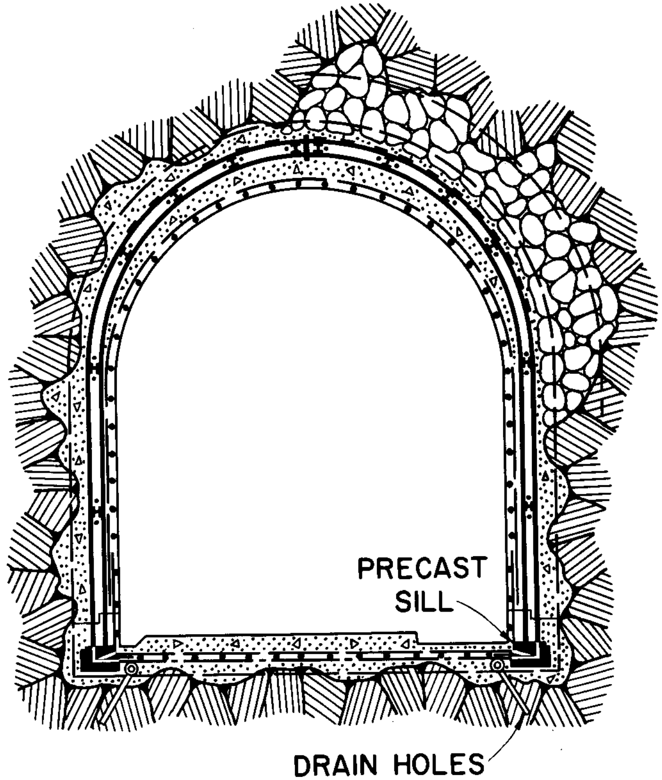


FIG. 5 TYPICAL SECTION-STEEL RIB SUPPORTED-SERVICE TUNNEL
HANABANILLA HYDROELECTRIC PROJECT

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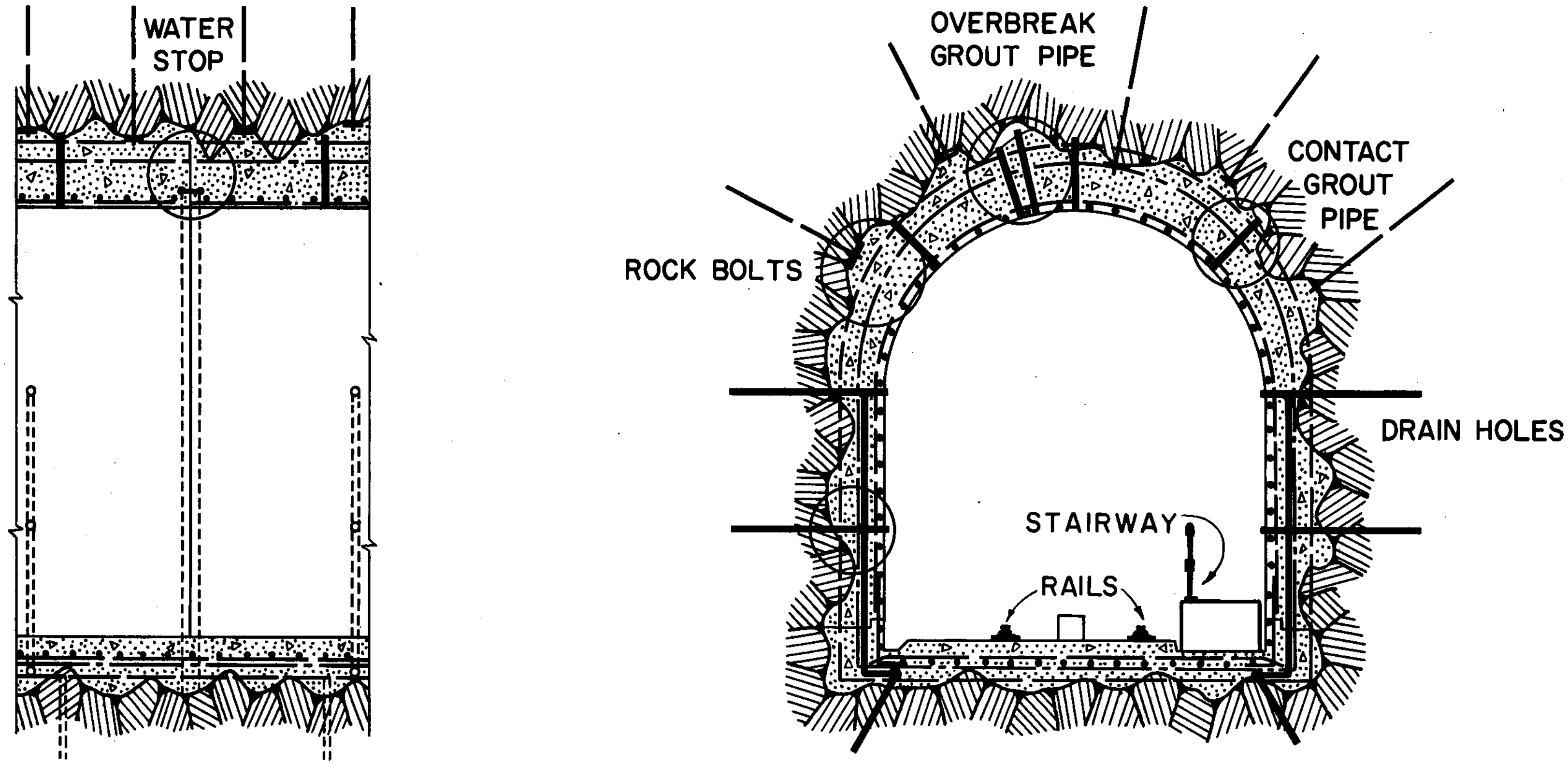

FIG. 6 TYPICAL SECTION - SERVICE TUNNEL - HANABANILLA HYDROELECTRIC PROJECT

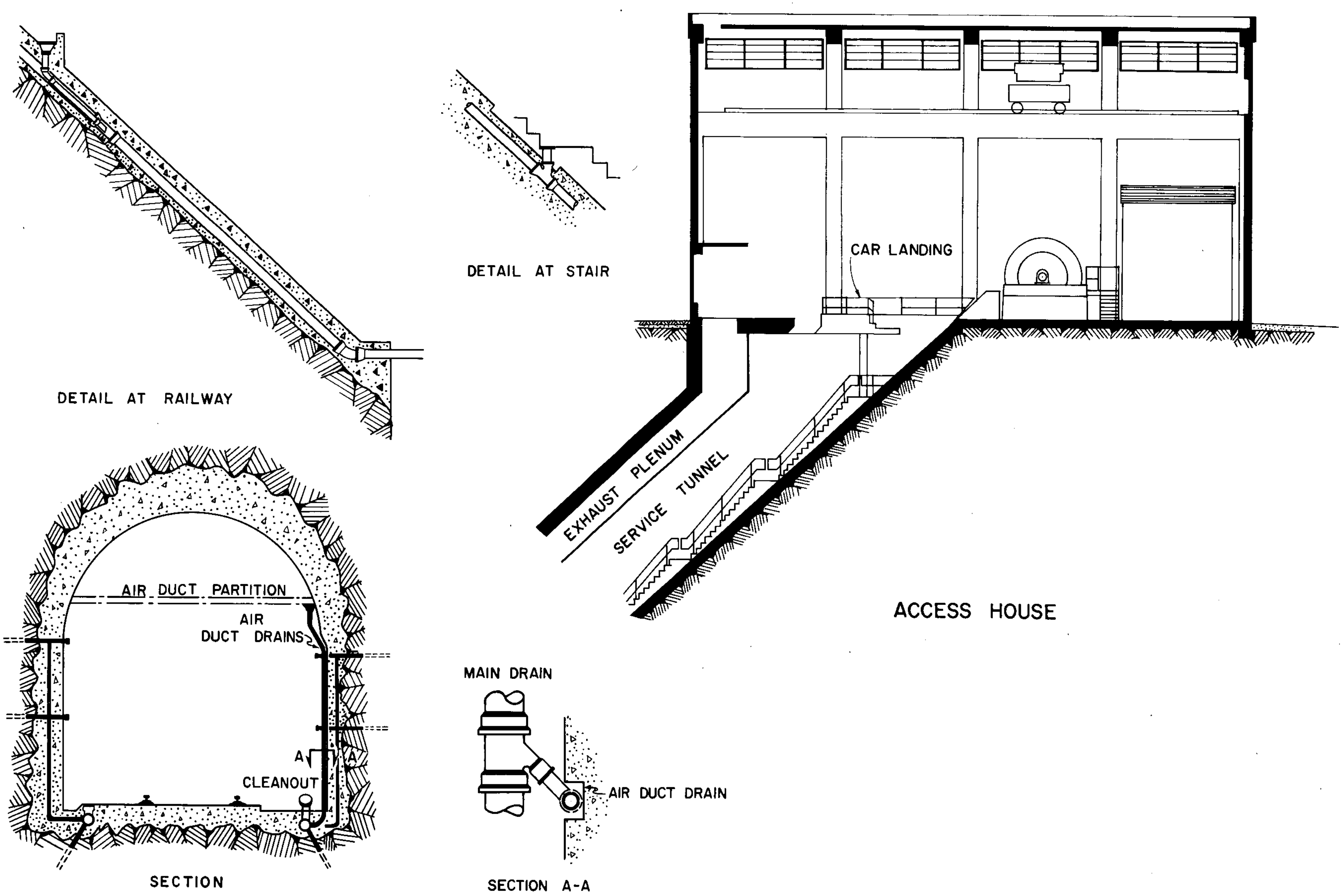

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FIG. 7 DRAINAGE DETAILS - SERVICE TUNNEL - HANABANILLA HYDROELECTRIC PROJECT

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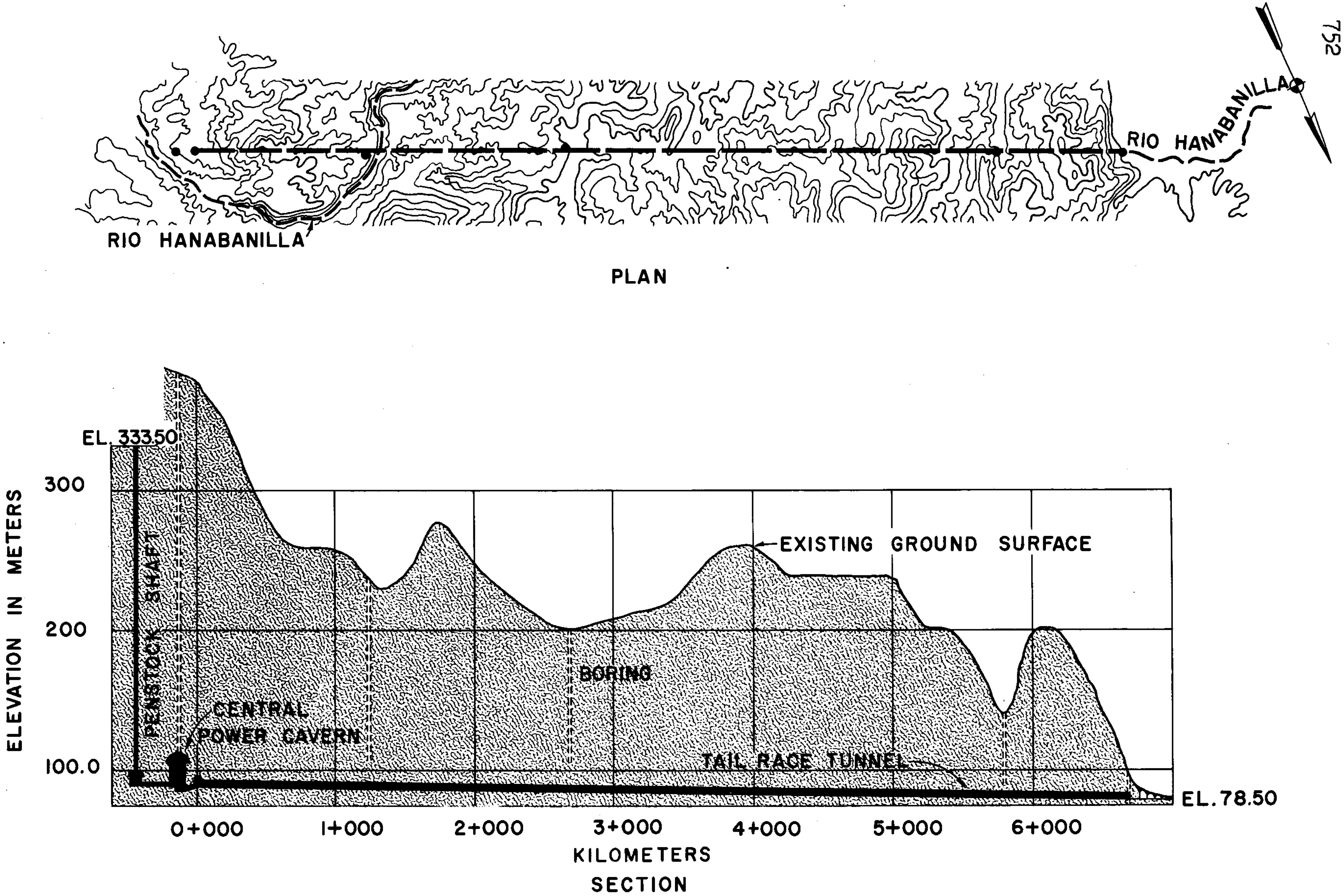

FIG. 8 LONGITUDINAL SECTION - TAILRACE TUNNEL - HANABANILLA HYDROELECTRIC PROJECT

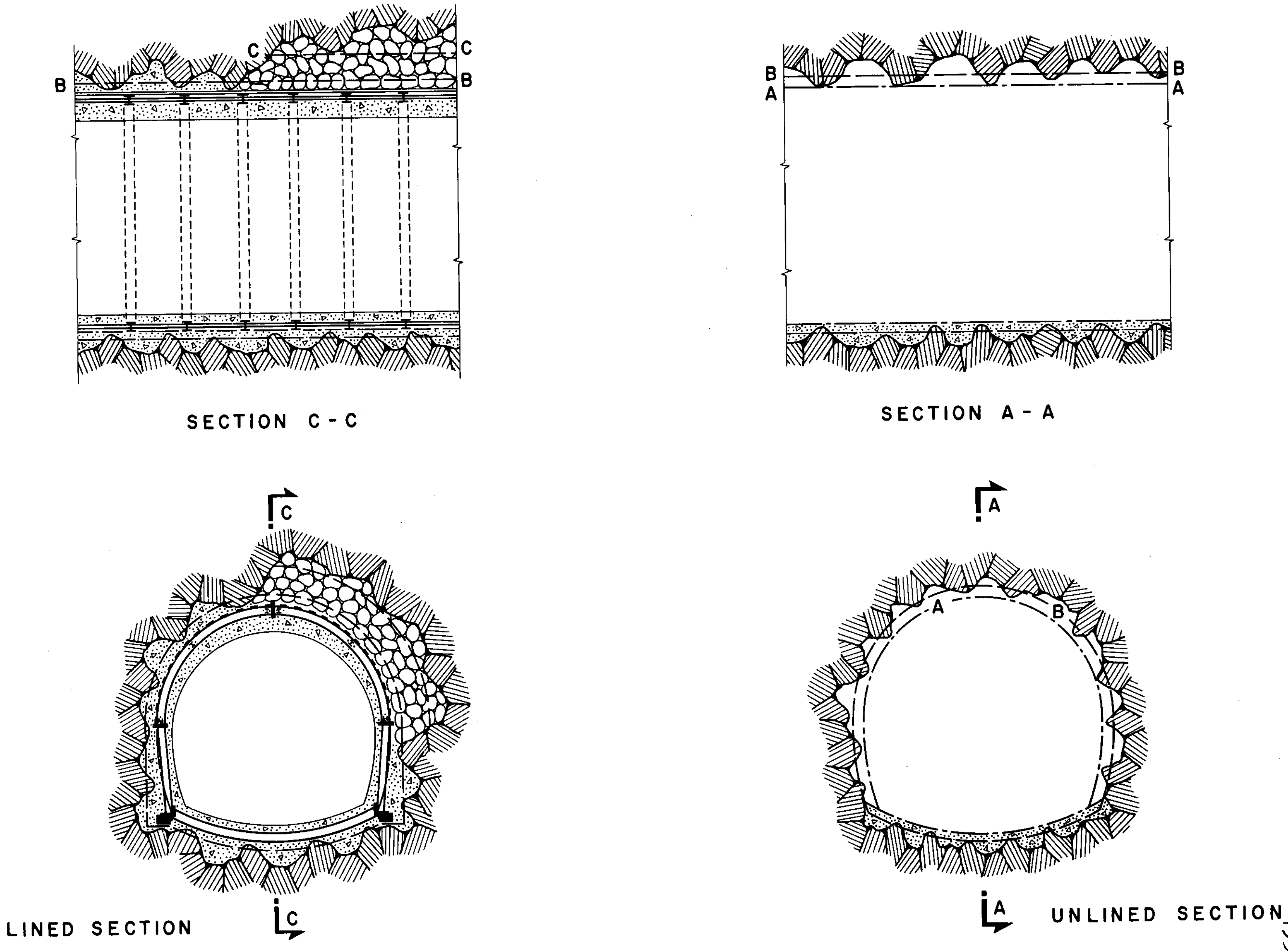

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FIG. 9 TYPICAL SECTIONS - TAILRACE TUNNEL - HANABANILLA HYDROELECTRIC PROJECT

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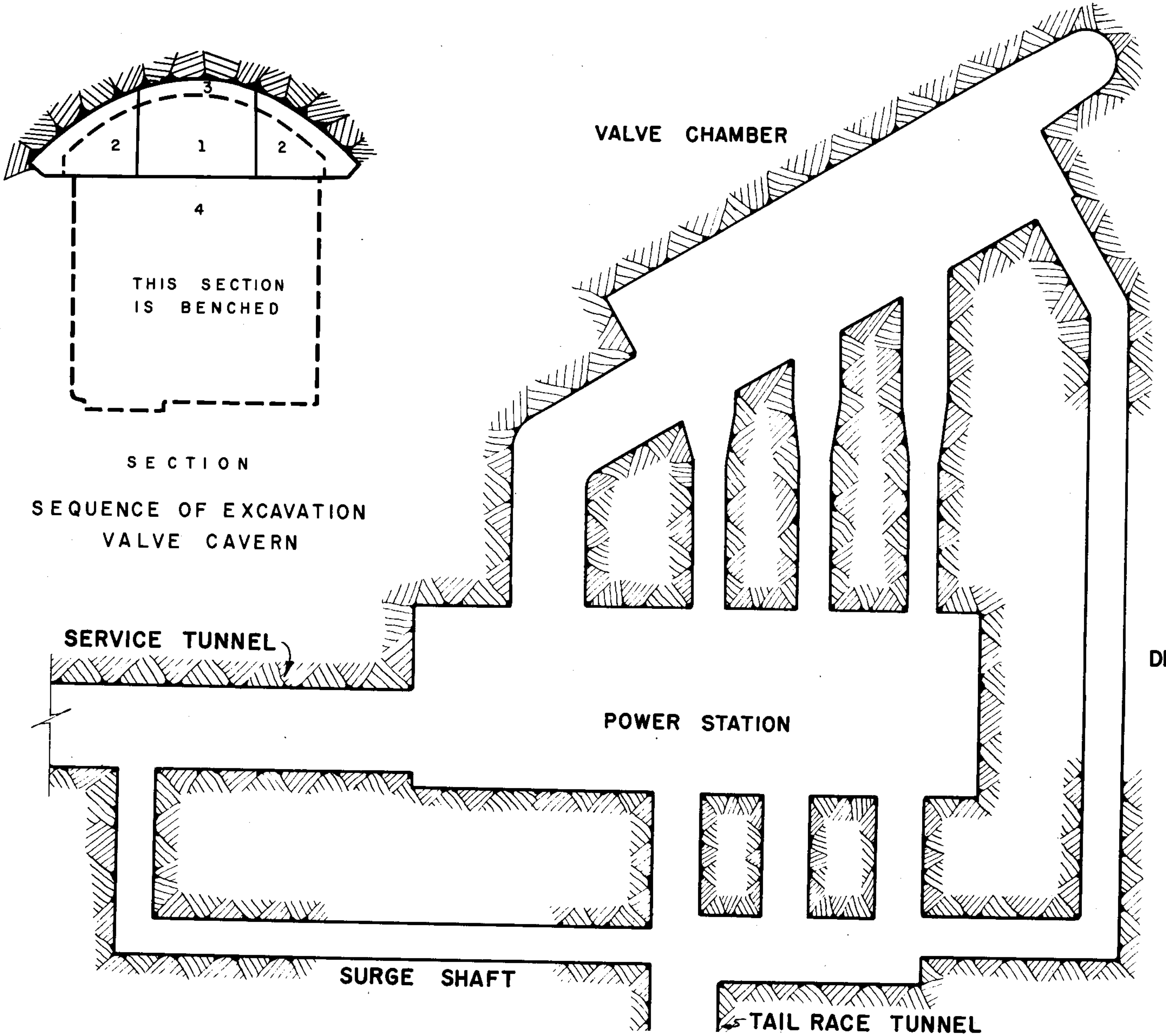

FIG. 10 PLAN OF UNDERGROUND CAVERNS—HANABANILLA HYDROELECTRIC PROJECT

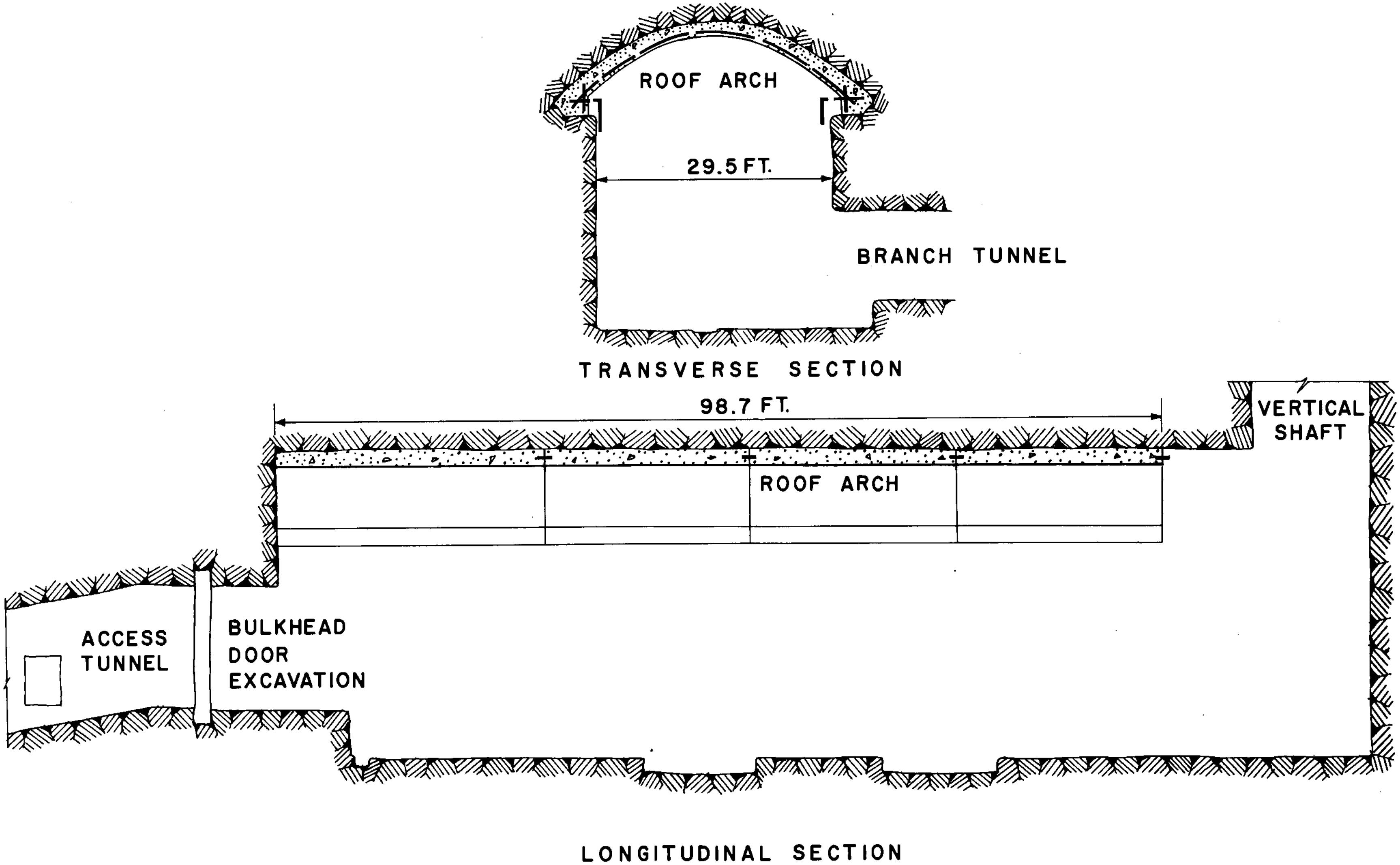

FIG. 11 SECTIONS - VALVE CHAMBER - HANABANILLA HYDROELECTRIC PROJECT

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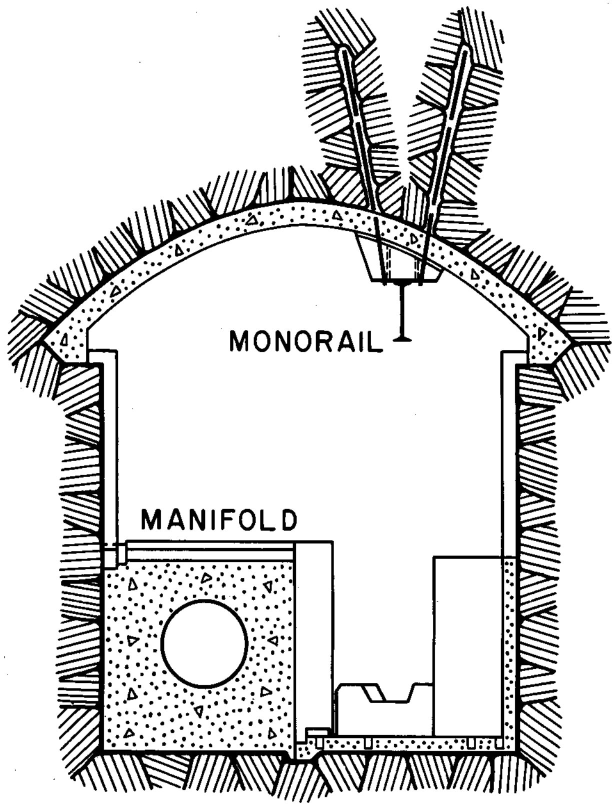

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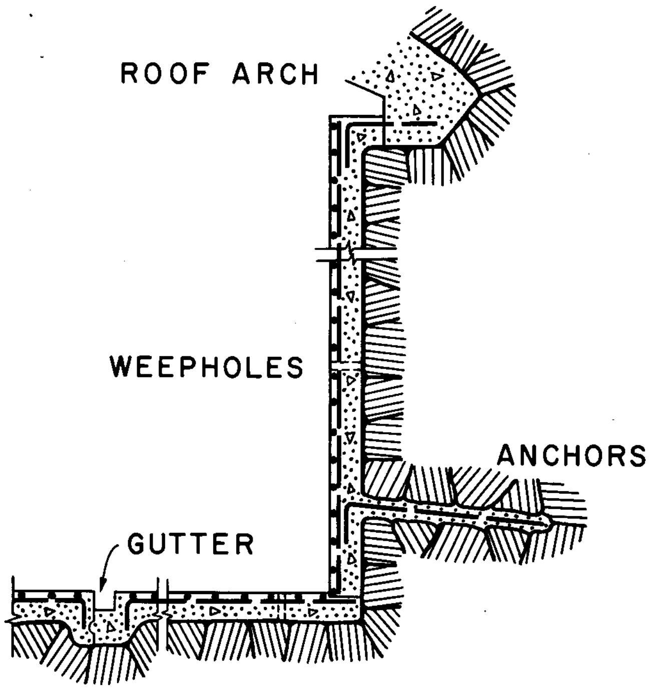

FLOOR AND WALL LINING

FIG. 12 DETAILS-VALVE CHAMBER-HANABANILLA HYDROELECTRIC PROJECT

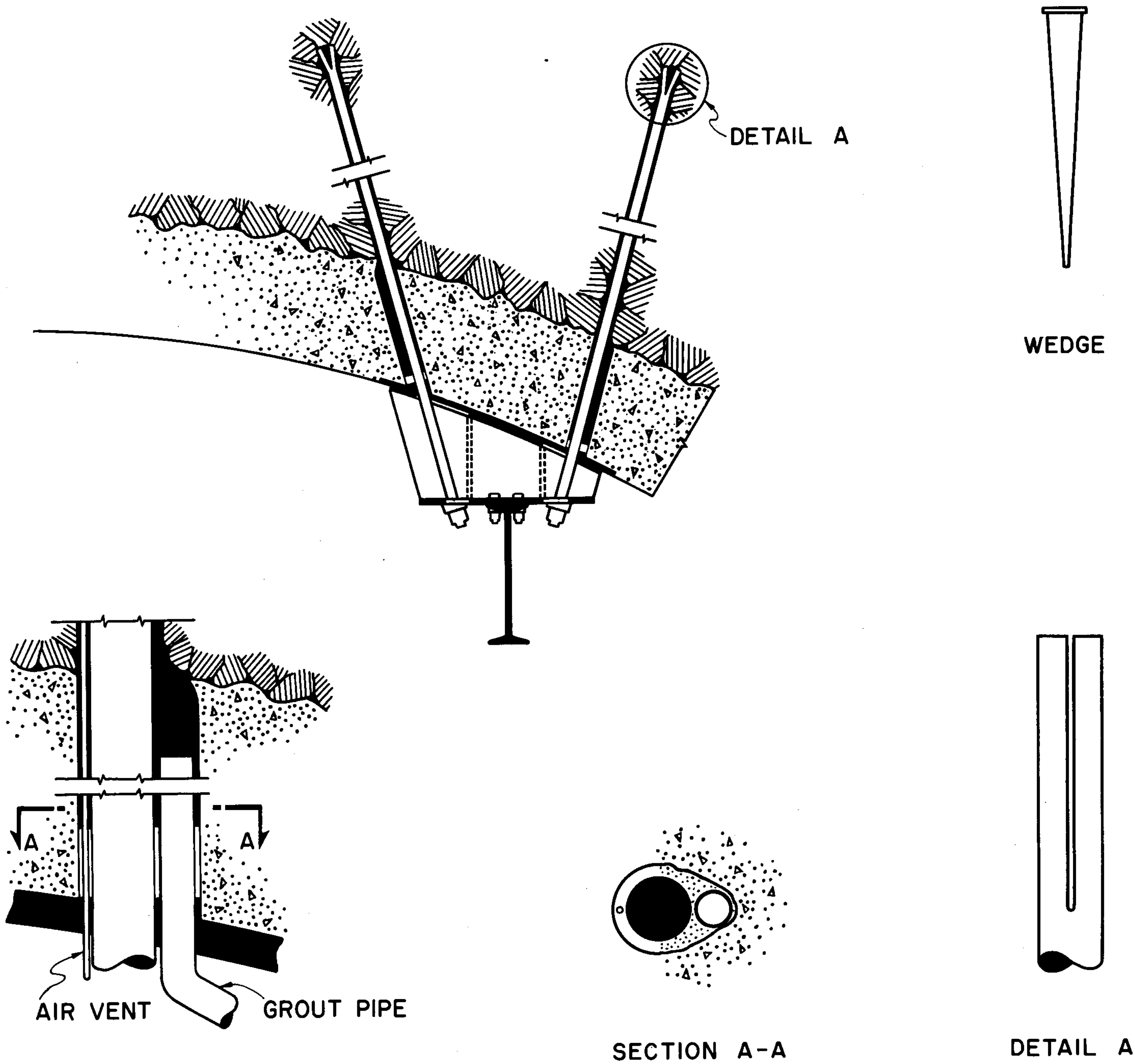

FIG. 13 DETAIL-VALVE CHAMBER MONORAIL
HANABANILLA HYDROELECTRIC PROJECT

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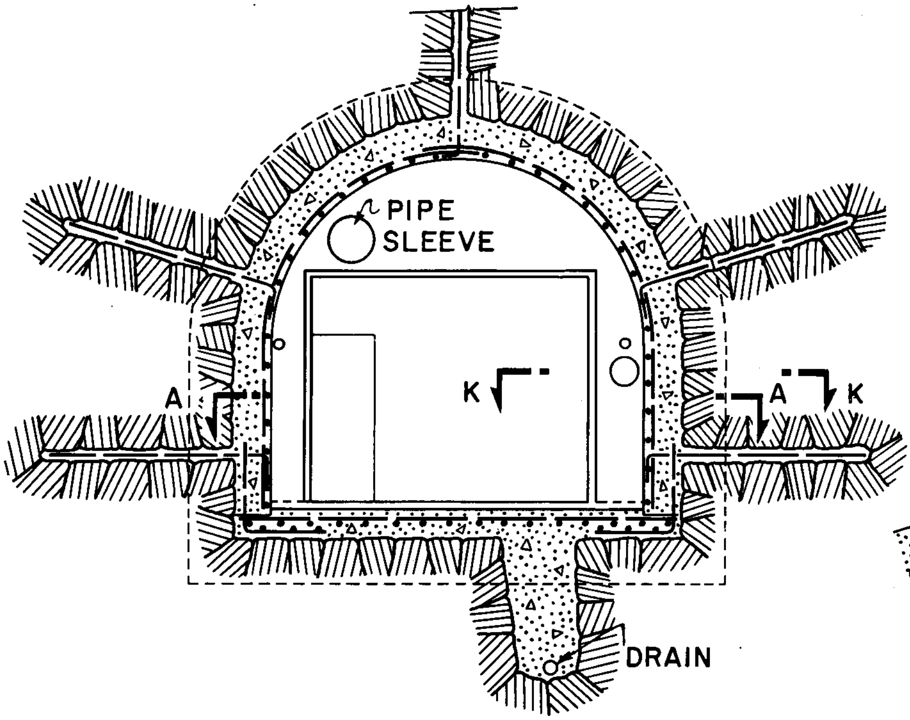
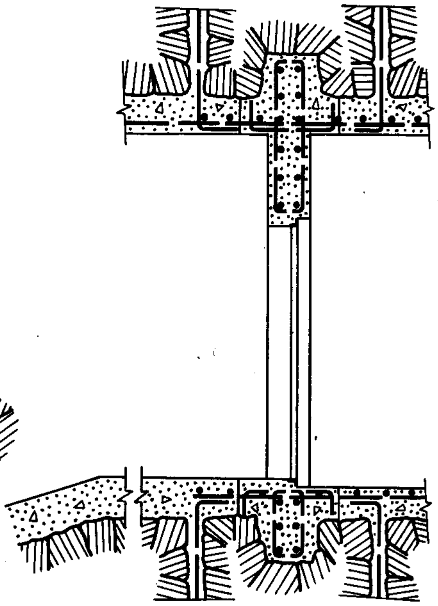
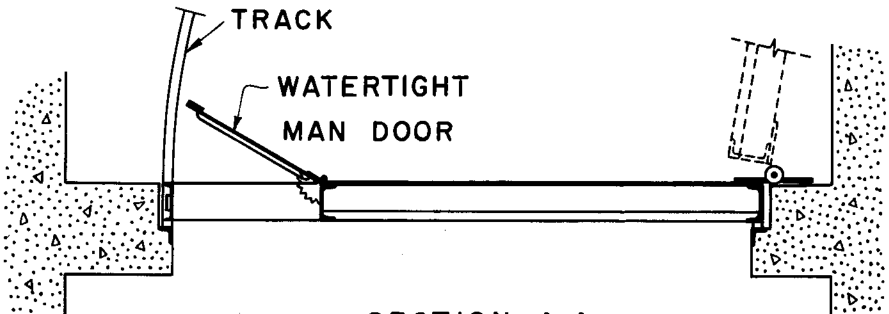


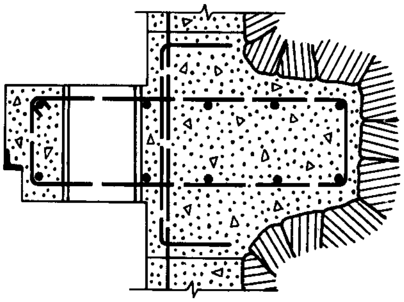
TRANSVERSE SECTION



LONGITUDINAL SECTION

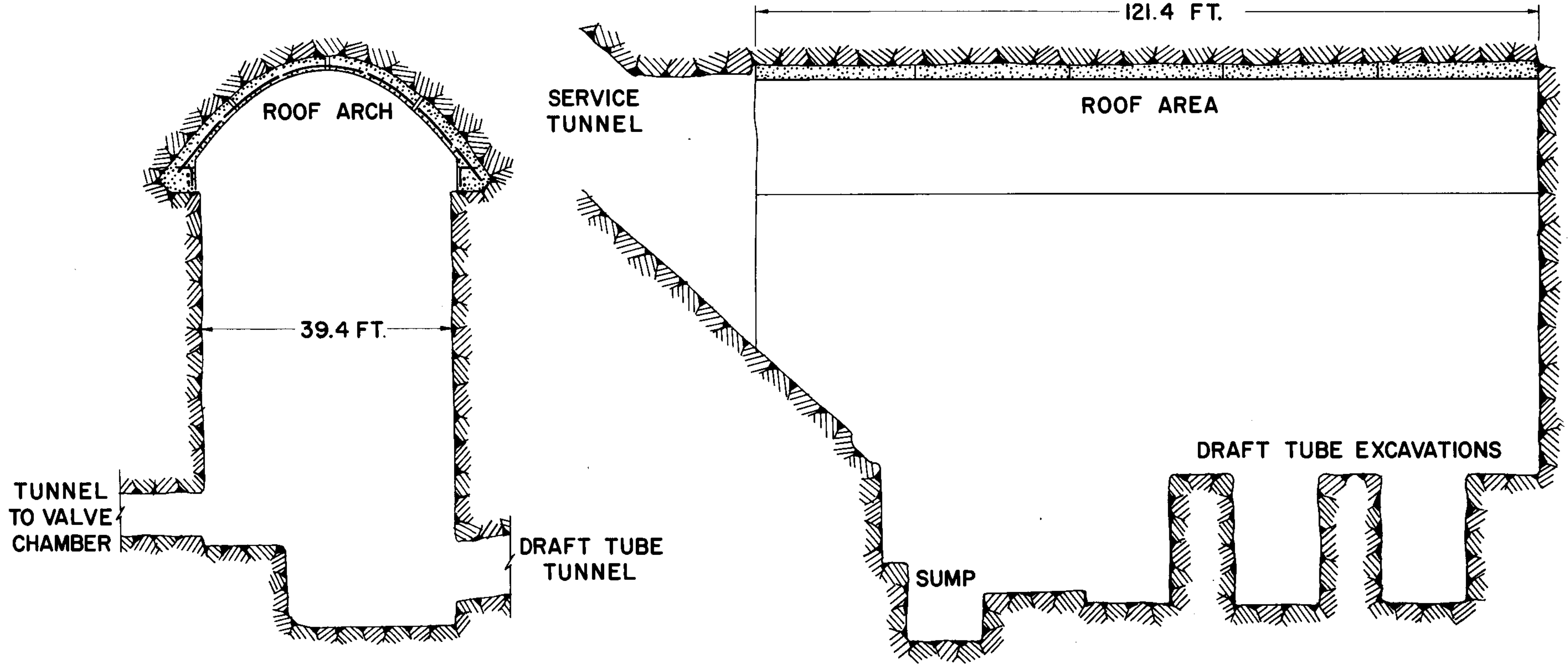


SECTION A-A



SECTION K-K

FIG. 14 BULKHEAD DOOR - ACCESS TUNNEL - HANABANILLA HYDROELECTRIC PROJECT



TRANSVERSE SECTION

LONGITUDINAL SECTION

FIG. 15 SECTIONS - POWER STATION - HANABANILLA HYDROELECTRIC PROJECT

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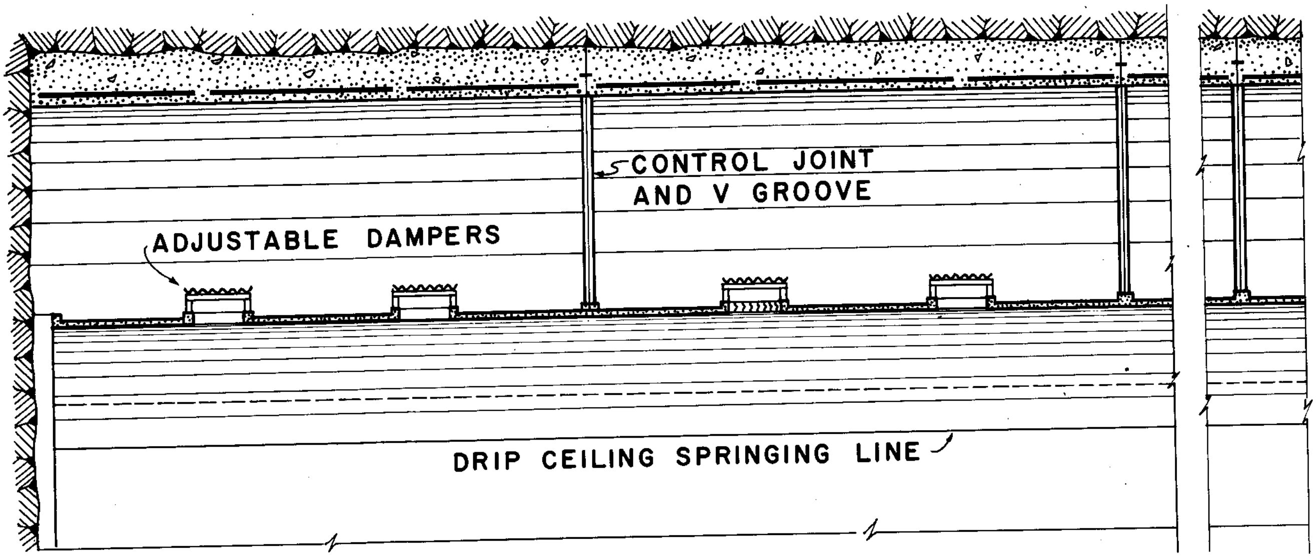
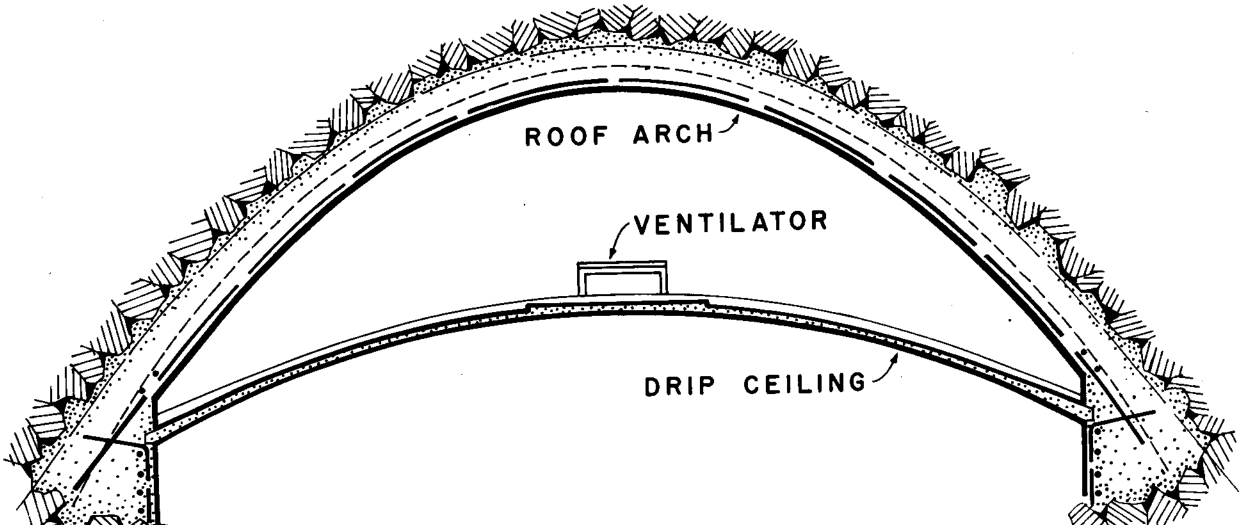


LONGITUDINAL SECTION



TRANSVERSE SECTION

FIG. 16 DETAILS OF ROOF ARCH AND DRIP CEILING-POWER STATION-
HANABANILLA HYDROELECTRIC PROJECT

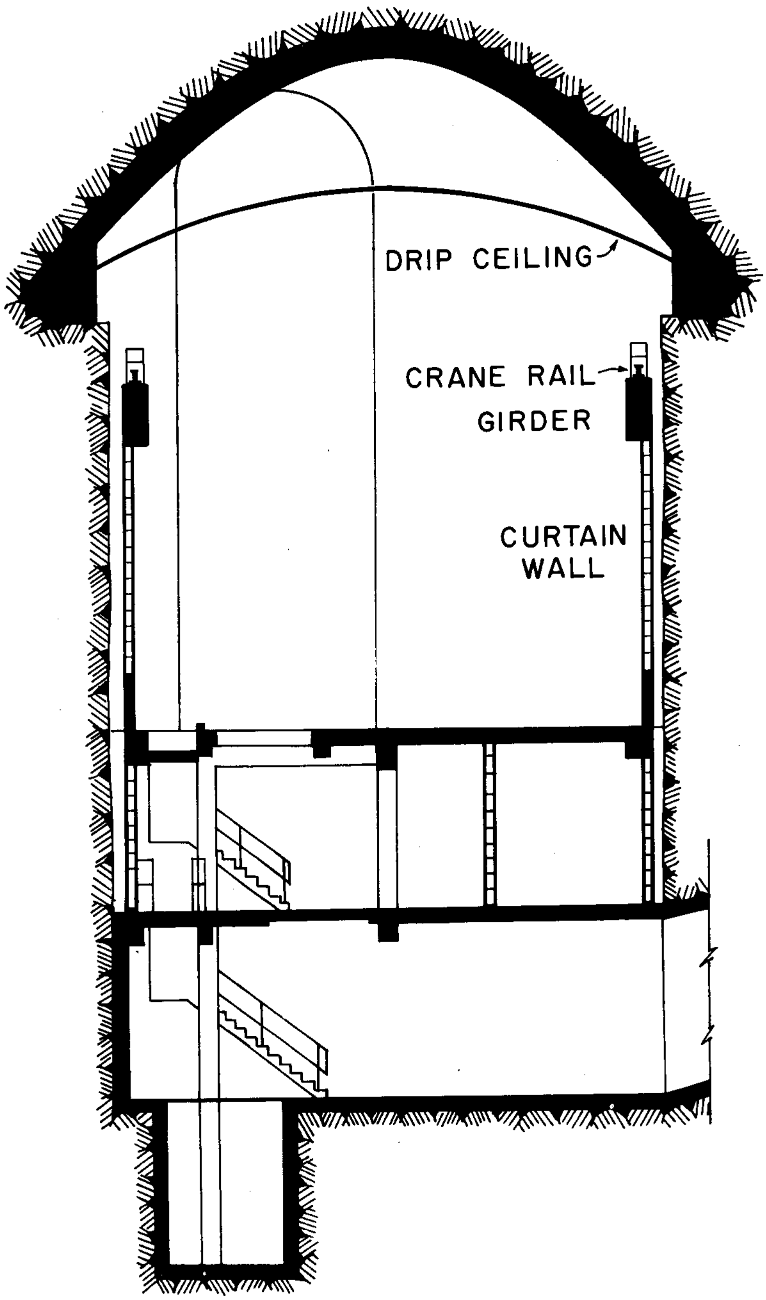


FIG. 17 WALLS - POWER STATION - HANABANILLA HYDROELECTRIC PROJECT.

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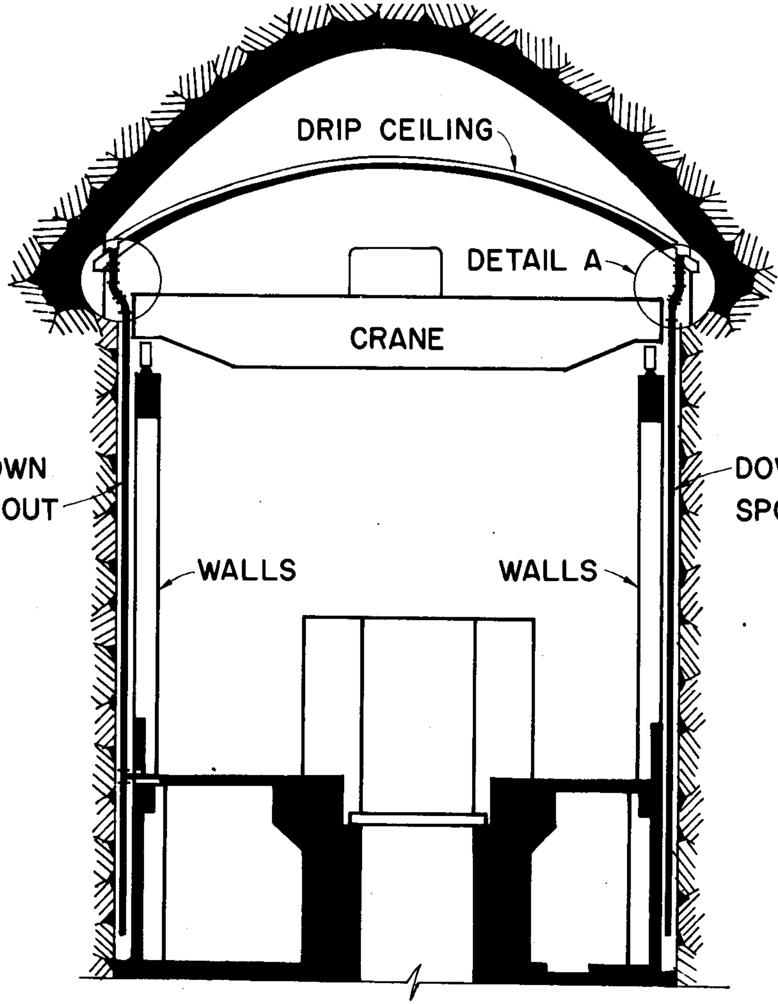
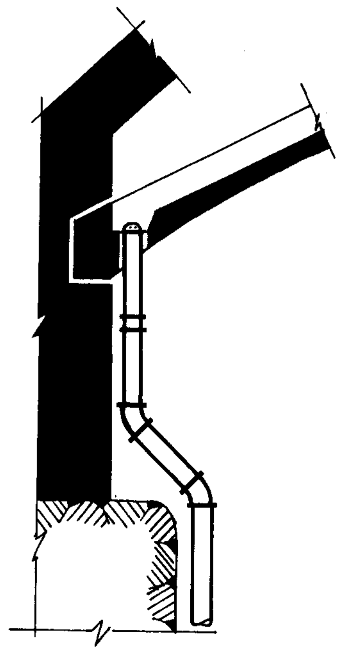


SECTION THROUGH POWER CAVERN



DETAIL A

FIG. 18. DRAINAGE FROM DRIP CEILING - POWER STATION - HANABANILLA HYDROELECTRIC PROJECT

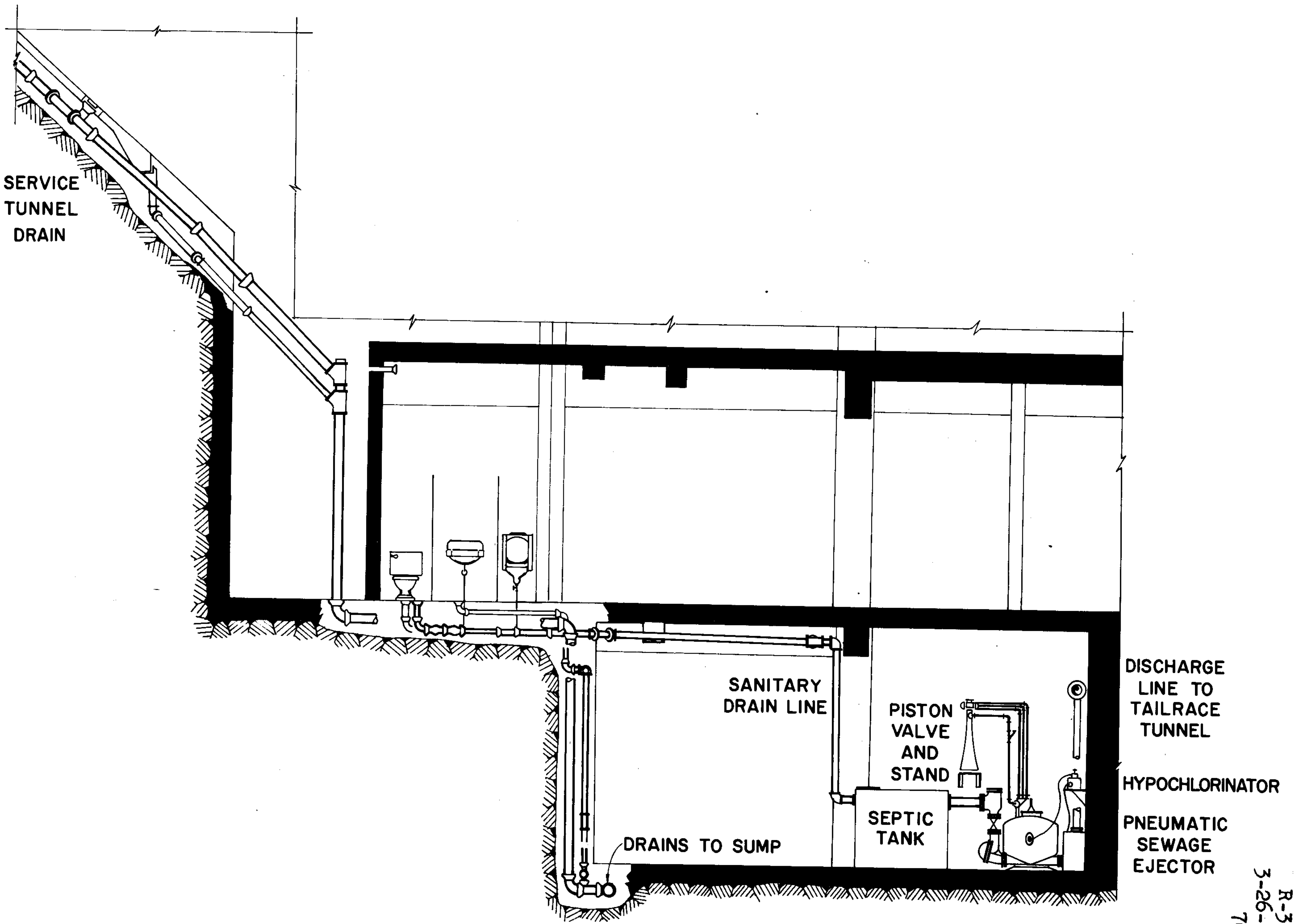


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FIG. 19 SEWAGE SYSTEM - POWER STATION - HANABANILLA HYDROELECTRIC PROJECT

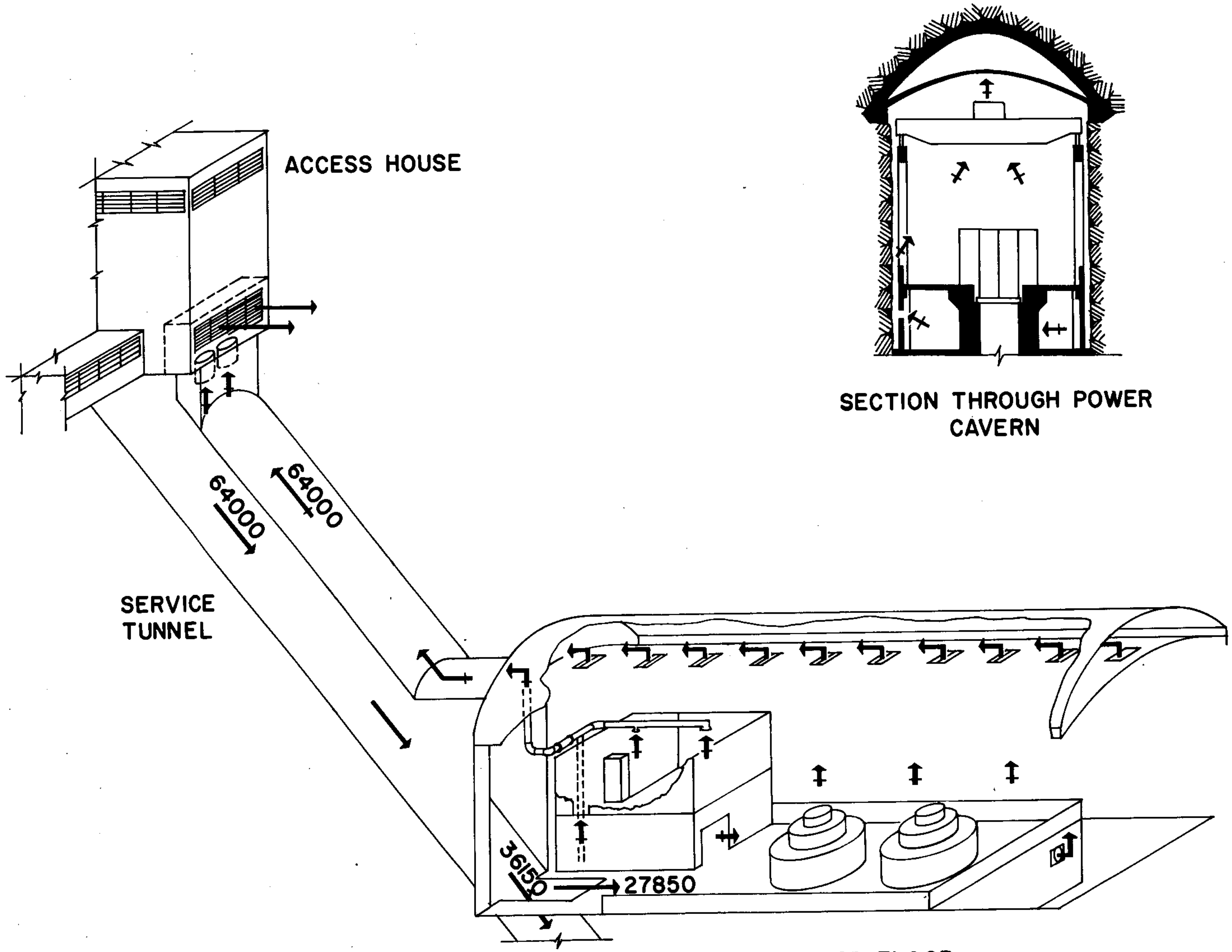

FIG. 20 VENTILATION - POWER STATION - HANABANILLA HYDROELETRIC PROJECT

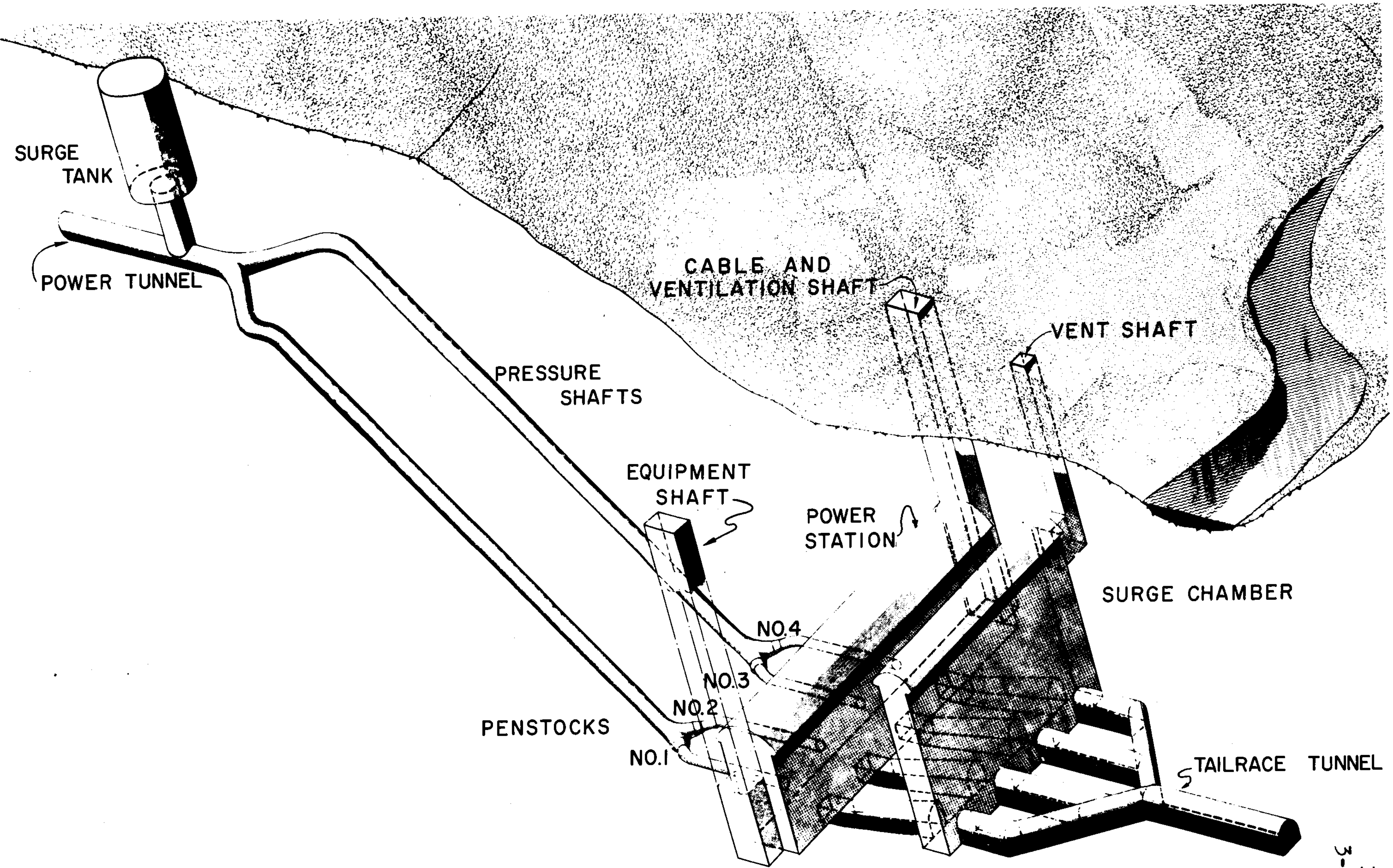

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FIG. 21 CAVERNS AND TUNNELS - BINGA HYDROELECTRIC PROJECT

UNDERGROUND EXPERIENCE IN THE SNOWY MOUNTAINS - AUSTRALIA

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INTRODUCTION

The Snowy Mountains Hydroelectric Authority is carrying out a large hydroelectric development in the Snowy Mountains which are located in the Southeastern part of Australia (Fig. I). The works will comprise seven major dams, some 100 miles of large tunnel, at least five underground power stations, surface power stations, and allied facilities. Descriptions of the scheme and details of various phases of the works have been published elsewhere (1, 2, 3, 4, 5). This paper is confined to investigations and construction experience associated primarily with the T-1 Power Station which will be used to illustrate a number of the principles involved in large underground works.

Underground Works

In the investigations, design, and construction of underground works the objective is to obtain openings of the required size which are permanently stable and adequate under the specified conditions of usage. In general, works may be grouped according to use, viz, (a) ingress; (b) egress; and (c) accommodation.

The first two are generally provided for by shafts or tunnels, whilst the third requires larger openings, such as halls, chambers, cells, vaults, or other open spaces. Many problems in design and construction are common to all three, but the problems associated with the larger openings in the rock required for accommodation purposes are generally more complex and difficult than those for the smaller openings of tunnels and shafts. Operation and maintenance of underground installations can also pose special problems.

INVESTIGATIONS

General

Investigations to provide basic information for design and construction are the first stage in the accomplishment of any underground work. The extent to which investigations should be carried out depends upon the purpose for which the works are intended, the magnitude and importance of the undertaking, what can be afforded in manpower, money, and time, and an objective and critical appraisal of all available data--including data accumulated as the investigations proceed.

The decision to "freeze" the investigations and design pattern and proceed with construction is a crucial one, and should be made only after making a realistic assessment of the ill effects, if any, which could result if further investigations are not made.

Snowy Investigations

When the Snowy Mountains Authority began its operations in 1949 little was known of the Snowy Mountains area (Fig. I), and a comprehensive mapping and investigation program was planned and progressively carried out. In view of the urgency for production of power and consequently the need to get construction under way as soon as possible, all types of investigation from broad regional surveys to detailed investigations of individual features, such as dam and power house sites, were undertaken simultaneously.

For topography the usual methods of aerial and ground survey were used and a first order precise survey net established for over-all control. Photo-theodolite surveys were used for details of feature sites and, in the steep rugged canyons which characterize much of the topography, proved to be very much faster and more accurate than other survey methods in such terrain. They were also used to record excavation progress-both surface and underground. Surface geology, both regional and local, was carried out by usual geological survey methods. Sub-surface investigations included diamond core drilling, seismic prospecting, and investigation shafts and tunnels. Owing to their high cost the latter were undertaken only after the other methods had established that a site appeared to be reasonably competent and construction feasible.

From the beginning of the work it was obvious that the behavior of rock in large tunnels and underground excavations would be a most important factor in design and construction. This led to a program of investigations relating to rock behavior and rock support, some aspects of which in connection with the T1 Power Station are described in this paper.

T1 Power Station

The region in which the power station is situated is a plateau, some 5,000 ft. above sea-level, which is deeply dissected by the Tumut River and its tributaries. In the vicinity of the T1 and T2 power stations the Tumut Valley is 2500 to 2000 ft. deep, (Fig. 2), and the river flows as a continuous series of rapids through a boulder filled channel, 70 to 120 ft. wide, usually without a flood plain.

Near the power station, the valley walls rise from the river channel on a slope of about 40^o to 45^o for a height of 1300 ft. and then flatten.

The power station is situated under the steep right bank, 1100 ft. in from the river bed at its nearest point and 1,000 ft. below natural surface.

The layout of the T1 Power Station is shown in Fig. 3 with details in Figs. 4 and 5. It has a capacity of 320,000 kw in four machines developing a head of 1,100 ft. The general dimensions of the machine and transformer halls are given in Fig. 4. The former has a maximum height of 105 ft. and the latter 43 ft. Vehicular access is through an access tunnel about 1,300 ft. long on a down-grade of 12-1/2 per cent. Personnel access is via a concrete-lined, vertical elevator shaft, 10 ft. in diameter and 1,200 ft. in depth.

The tailrace surge chamber is an enlarged tunnel section some 1,000 ft. in length with a horse-shoe shaped upper chamber 25 ft. high and 220 ft. long. From the tailrace surge tank a tailrace tunnel 3,300 ft. long and 25 ft. in diameter carries the water back to the Tumut River.

A number of sites in this area were examined progressively by surface mapping, air-photo interpretation, and diamond drilling. An assessment of these results led to the selection of the present site which appeared to be free from major faulting and to be the most favorable.

The site was explored initially by geological mapping of the surface and by diamond drilling. Five sloping holes from 705 ft. to 2005 ft. in length were drilled from the surface, four of them extending to below the level of the station. Based on this drilling a site for the station was tentatively adopted and explored further by a tunnel approximately 8 ft. by 8 ft. in cross section and 1100 ft. long driven from the Tumut River into the station site. From chambers near the end of this tunnel 6 diamond drill holes with a total length of 1377 feet were drilled into the machine hall area and across the tailrace surge chambers.

The rock at the power station site is of two types (1) Boomerang Creek gneiss and (2) Happy Valley granite. The granite occurs in sheet-like masses 100 to 300 ft. thick intrusive into the gneiss. Their distribution at power station level is shown in Figs. 3 and 6. The contact between the two rock types is usually gradational over a distance of several inches but in some places the transition zone is as much as 20 ft. wide. Viewed broadly the boundaries are fairly straight for distances of several hundreds of feet, but in detail they show many irregularities.

Several dolerite dykes varying up to several feet in width occur in the access tunnel and pressure shafts, and a small one in the machine hall roof.

On the steep slopes above the power station there are numerous scattered rounded outcrops of broadly jointed granite forming low cliffs, and less conspicuous typically angular, closely jointed outcrops of gneiss. The outcrops are much loosened by mechanical weathering, and many blocks are on the verge of instability. Weathering may extend from 100 to 350 ft. below the surface.

Occasional large masses of granite appear almost fresh in outcrops, but often prove to be huge residual "boulders" or pinnacles of fresher granite surrounded by more intensely weathered granite. The depth and intensity of weathering is generally least in the valley floor and increases up-slope.

Both rock types at the power station site are extensively jointed. Although this jointing is not developed in a very regular pattern, it is possible to distinguish three principal directions of jointing which are approximately parallel to the minor faults A, B and C shown in Fig. 6.

The two rock types exhibited characteristic differences in the general spacing of the joints and the nature of the joint surfaces. The joint spacing in granitic gneiss was usually between 6 ins. and 2 ft.

The joint surfaces were usually smooth and slickensided and dark green in color due to thin coatings of chlorite. Clay coatings were not common and most joints were tightly closed. Because of the nature of the joint surfaces it was common in the excavations for the granitic gneiss to break to the natural joint planes rather than break across the rock resulting in angular surfaces and local small overbreak in many places.

The joint spacing in the granite was variable but was generally in the range of 1 to 5 ft. The more closely jointed zones occurred irregularly, and slickensided joints were not so common as in the gneiss. Most joints were tightly closed, and because of this it was found that breakage of the granite often occurred along new fracture surfaces rather than along natural joints.

Another geological structure of importance was a nearly horizontal persistent zone of close fracturing, a few inches wide with heavy limonite staining along the fractures which occurred throughout the machine hall and transformer hall area, D in Fig. 6. It was located a short distance above the portals of the penstock tunnels on the upstream wall, and above the tailrace tunnels on the downstream wall.

It was judged that the power station could be constructed in either rock type, but because of the difference in jointing it was considered that large openings would be easier to construct in the granite and would require less permanent support. For this reason the greater part of the station was located in the thickest granite sheet.

In determining the orientation of the power station, the directions of jointing and shearing were the most important geological considerations. In general it is undesirable to have persistent joints parallel or nearly parallel to a large surface, such as long high walls. The more closely the direction of strike of such joints approaches the direction of the normal to the surface, the less effect the joints have in causing instability.

Although the joint pattern predicted from the exploratory tunnel and on surface exposures proved to be quite similar to that from the actual power station excavation, the narrow, very persistent minor faults A and B, (Fig. 6) were not discovered during the exploratory stage. On the other hand, shearing which was noted in several localities at the boundary between granite and gneiss did not occur in the actual excavations.

Large flows of groundwater were struck during drilling from the end of the exploratory tunnel, and reached a maximum of about 800,000 gallons per day. The peak flow from the access tunnel and power station excavations including contributions from the exploratory tunnel, drill holes, pressure shafts, and lift shaft, was about 1,800,000 gallons per day, which is remarkably low in view of the high flows encountered in the exploratory work.

As the excavations progressed it was observed that most of the rock mass was practically impervious owing to the joints being tightly closed or sealed. However, a few joints were encountered which were open, and practically the entire groundwater inflow came through these fissures which had been intersected by the exploratory drill holes.

The free draining properties of the rock mass have been preserved by limiting the grouting of the rock around excavations.

In view of the groundwater conditions, it was considered that both the roof and the walls should be free draining, and an open rib type of roof with 4 ft. by 4 ft. reinforced concrete arch ribs at spacings varying between 8 ft. and 12 ft. was adopted for both the machine hall and the transformer hall, (Figs. 5 and 7). At the roof abutments the ribs are supported on a reinforced concrete beam which is continuous except for contraction joints spaced at approximately 100 ft. centers.

To expedite construction and for economy, it was decided to use rock bolts to support the roofs of the machine hall and the transformer hall prior to placing the concrete ribs, and also for wall support. (Figs 7 and 20).

Above generator floor level on the upstream wall of the machine hall and above the main floor level at each end wall, the design provided for the vertical rock walls of the power station to be free draining. The location of the draft tube gate slots (Fig. 7) necessitated placing concrete against the rock walls, and weep holes into these slots are provided.

Moye (5) has given a detailed account of the investigations, geology and rock properties for the T1 site. Average results for mechanical tests on the two types are given in Table 1.

The most significant feature of the test results was the high compressive strength. This was approximately 20,000 psi for both types except for the gneissic granite (Type 2) in the transformer hall area where it was 14,000 psi. The rock also had a definite limit of proportionality in compression at approximately 12,000 psi. Poisson's ratio was about 0.25.

STRESS IN A ROCK MASS

Natural State of Stress

The natural state of stress in a rock mass is here used to define the state of stress in the rock mass prior to disturbance by the excavations.

The assumptions most frequently made when attempting to assess the natural state of stress which may be encountered in large underground cavities are the ideal ones that the rock is homogeneous, isotropic, perfectly elastic, and that nowhere do the stresses exceed the elastic limit of the rock, (4), (5). It is rarely, if ever, that these ideal conditions are realized and estimates for the stresses at the site of underground excavations must take into account geological structure, mechanical and chemical state of the rock, and topography. Also the assumption that the initial stresses are uniform in the zone to be excavated is only justified where the excavation is distant from the natural surface.

The test results for unjointed and unaltered gneiss and granite at T1 showed they are strong rocks with quite similar strength and elastic properties. However, these are not necessarily the properties of the rock mass, which is disrupted by joints and faults and also by blasting during construction operations. Ideally the strength and elastic properties of the rock mass should be measured in place. However, in a rock mass in which the joints are tightly closed and interlocked the compressive properties may not depart very significantly from those measured on specimens in the laboratory. The tensile properties are likely to be very different since the rock mass in tension would certainly part readily along existing fractures.

A measure of the compactness or degree of interlocking of the joint blocks of the rock mass is given by the manner in which it breaks during excavation. In the gneiss at T1 the surfaces of excavations were composed almost entirely of flat joint faces, indicating that under the effects of blasting the rock mass broke more readily along joint planes than across joint blocks. In contrast, in the granite was a large proportion of these surfaces composed of fresh fractures where the rock mass broke across joint blocks rather than along existing joint planes.

If the ideal assumptions are made for the T1 site, then the vertical stress σ_z due to gravity would be

$$\sigma_z = \rho z \qquad (1)$$

where ρ is the unit weight of the rock and z is the depth below free surface.

Ignoring any tectonic forces or other disturbances, the horizontal stresses σ_x and σ_y in the rock corresponding to σ_z would depend only on the elastic properties of the rock. If lateral expansion is prevented and ν is Poisson's ratio, then

$$\sigma_x = \sigma_y = \frac{\nu}{1-\nu}\sigma_z \qquad (2)$$

Under these conditions σ_x, σ_y, and σ_z are the principal stresses; and N the ratio of horizontal stress σ_H to vertical stress σ_V is given by

$$N = \frac{\nu}{1-\nu} \qquad (3)$$

At T1 Power Station, where z is 1,100 ft., ν is 0.25, and ρ is 171 lb. per cu. ft.

$$\sigma_V = \rho z = 1310 \text{ p.s.i.} \qquad (4)$$

$$\sigma_H = \frac{\sigma_V}{3} = 436 \text{ p.s.i.} \qquad (5)$$

For a groundwater level 900 ft. above the station the groundwater pressure at the site would be 390 psi. This implies that joints in the rock may be held open by the groundwater pressure if σ_H were of this order. Open vertical joints carrying water were found at right angles to the machine hall axis.

The natural state of stress was partially determined at T. 1 power station and at two other localities in the Snowy Mountains, by measuring the tangential compressive stress in the surface layer of rock around the excavations in horizontal and vertical directions, using an adaptation of the flat jack method described by Tincelin (16). This work, which has been described by Moye (5) showed that approximately σ_V was 1,500 psi, and that σ_H was 1,700 psi normal to the machine hall axis and approached zero parallel to this axis. This latter result is consistent with the open joints c, Fig 6, at right angles to the station axis. It is obvious that designs based on equations (1) to (5) could be far astray and may be disastrous.

It is considered that actual measurements of the natural state of stress, even though the results must be used with caution, owing to the errors inherent in such measurements, is a prime essential for the intelligent design and construction of underground works.

The natural state of stress can also be disturbed by stress concentrations caused by a V notch like the Tumut Valley (Fig. 2). Qualitative photo-elastic studies using gelatin models were made of this effect and indicated that σ_H could be very much higher than given by equation (2). The past tectonic history of the region and the present state of tectonic activity may also influence the state of stress in a manner which cannot at present be reliably predicted.

Excavation Sequence

The actual state of stress following excavation of an opening depends not only on the initial stress condition in the rock, but also on the shape of the opening, the dimensions of the opening, and the geological conditions actually encountered. Many investigators (6, 7, 14, 15) have examined the stress concentrations for circular and elliptical tunnels and spherical cavities. These results are useful as a guide for normal tunnel work but must be used with considerable judgment in the case of a large irregular opening, such as a power station. In order to assess the conditions that might be encountered during construction of the T1 Station, photo-elastic studies were made of the proposed excavation sequence.

In the design of the T1 Power Station it was assumed that, in the machine and transformer halls, the section of the excavation from the crown of the roof to the abutments would be excavated first and that the concrete ribs would be placed before the excavation was carried down (Fig. 5). This would give a narrow elliptical opening with a major axis of some 77 ft. and large stress concentrations in the abutment areas. The effect that the progress of excavation would have on the stress distribution around an opening of the shape planned for the machine hall and transformer hall was also required. To obtain this information a series of photo-elastic studies, covering the various stages of excavation from the roof section to the complete opening were undertaken. These studies were also used to ascertain the effect that the ratio, N, of horizontal to vertical stress would have on stresses round an excavation of this shape. Typical illustrations from this sequence of studies are given in Fig. 8. Fringe values Fn and Fs for a σ_V of 1,000 psi are given with each figure. Fn is the value of normal stresses and Fs the value of shear stresses. Studies were also made with stress field at 45° to the vertical axis of the power station. Stress concentrations in terms of fringe numbers at important points on the boundaries of the excavation are marked on Fig. 8 and an indication given of the tension and compression zones. The series of tests illustrated by Fig. 8 used gelatin models so as to obtain the effect of body forces for various values of N, and illustrates the elimination of the tensile zone in the roof and floor with increase in N.

These studies showed that the worst conditions would occur when the first stage of excavation was opened up. Maximum concentrations occurred at the abutments of the roof and, providing the rock in the roof was stable under this condition, it should be satisfactory as the excavation proceeded downwards. These tests also showed that it was possible to have a tension zone in the rock away from the surface in the vicinity of the abutment beams.

Studies were also made for various other methods of opening up the roof zone. These showed that opening up the whole area of the roof section from the crown to the abutment level, probably gave the most suitable conditions. Other photo-elastic studies were used to plan the relative location and orientation of the various openings in the T 1 area.

Effects of Blasting

The depth of penetration of the effects of blasting - loosening of the surface layer of rock - is not known, although attempts are being made to measure it by a seismic method.

Blasting applies a sudden impulsive increase in pressure in the vicinity of the charge which results in fracture of the rock and its displacement. This is accompanied by both pressure waves and vibration in the rock mass. The first impulsive increase in compression is followed by a complete release of stress normal to the new surface. It also may release pressure water in the joints. This, in effect, gives a sudden elastic strain and redistribution of stress around the opening, which is followed by a time effect. The relaxation of either strain or stress or both with time is well known (8), (9), and is the explanation of what miners and tunnellers call the "bridge effect" or "hang-up" time.

ROCK JOINTS AND ROCK FAILURE

The results of theoretical analyses or of photo-elastic studies must be used with caution for structures such as rock excavations, particularly underground excavations. Rock as a structural material is neither homogeneous, iostropic, nor perfectly elastic and, is often looked on as a brittle and rigid material that fails only by "fracture". However, both igneous and sedimentary rocks when subjected to high temperatures or pressures or both tend to "flow" and take on the nature and properties of plastic or viscous materials (8), (9).

Failure by "flow" is connoted by such mining terms as "squeezing" or "swelling rock" and although such rocks have been controlled by rock bolts or anchors they are not dealt with in this paper.

The rock failure under consideration can be defined as due to "slipping", "separation" or both, and is generally closely related to the joint pattern in the rock.

Slipping includes fracture by shear through the intact rock or by slipping on joint planes. In the latter case, not only must the surface friction of the joint be considered but also any interlocking between the joint surfaces which would result in shear also playing a part.

Failure by a separation fracture can be caused in two ways, viz, tension or rotation. Tension failures either direct or caused by bending are well recognized on a large scale by geologists. "Popping" rock is another form of tension failure. Failure by rotation is generally the direct result of the joint system in the rock. Under the combination of forces which may exist in the rock mass individual blocks are displaced. The displacement can be either a simple slipping or sliding, or in the more general case, the block is rotated under a moment, the joint opens and failure occurs by slipping or by crushing or shearing at the corners of the block. This is the typical action of "stoping" which is the familiar term used by tunnellers and miners to indicate a process of fall-out from the roof of an excavation. Another form of failure "spontaneous disintegration", which is found only rarely, was exhibited by basalts in the T1 area which, after exposure varying from a few weeks to a year, disintegrated into pea-size pieces.

It is very rarely that rock free from joints is found in nature. Joint patterns are never random, they may be irregular or regular. Joints may be smooth surfaces, either plane or slicken-sided, but most joints are either rough or have irregularities, and the irregularities on one joint block very commonly interlock almost perfectly with the complementary irregularities in the adjacent joint block. Even polished slickensided joints which are smooth in the direction of the slickensides are generally irregular in all other directions. Under these conditions the force acting in the surface of the excavation must be great enough to overcome not only the friction of the joint surface but also to shear off the interlocked irregularities if the block is to slide. These conditions apply to very many joints.

If the joint blocks which are separated by a rough joint can move apart or rotate, the amount of interlocked rock is reduced and less force is required to cause failure of their interlocked rough joints and to permit movement along the joint.

Considering a group of joint blocks, movement may also be prevented or limited by the interlocking effect of adjacent blocks. For failure to occur it may be necessary for interlocked parts of blocks to be sheared or crushed. In general, any jointing pattern is not persistent over great distances. However, as in the case of T1 Power Station, persistent joints were found, for example (A), (B) and (D) in Fig. 6. In such cases the joint is generally undulating. Joints may also be lubricated by the accumulation in the joint of debris from rock movements, depositions from circulating water, or the end products of the chemical decomposition of rock minerals.

It is obvious that jointed rock is neither homogeneous nor isotropic and in the mass may be only moderately elastic. In some areas, it may be completely non-elastic and exhibit all the properties of a plastic or viscous material. The joint system may range from joints at wide intervals of many feet to closely spaced joints of a few inches. Although diamond drilling and exploratory tunnels give much valuable information, the detailed condition of the rock is not known until the excavation is made. The materials which may be encountered can range from almost intact rock to a material that resembles crushed rock aggregate. In the latter case although the size of the fragments may be comparable to aggregate sizes, they never have the random arrangement found in aggregate.

However, the fact is that rock is jointed and must be dealt with as such if a satisfactory structure is to be built.

In general it is known that the jointed rock around an excavation is loose to varying degrees, and that the rock mass appears to fail by sliding in towards the excavation along joints, accompanied by rotation of blocks, by crushing of irregularities along the joint planes, and by shearing or crushing of parts of interlocked blocks. These slight movements permit the outer layer of rock to become wholly or partly destressed, and the highly stressed zone migrates back from the surface of the excavation until eventually the rock mass becomes strong enough to carry the load. It can be inferred that this is brought about by a decrease in the difference in the minor and major principal stresses and by the accompanying increase in the strength of the rock under triaxial conditions. In this manner a "decompressed" zone develops around an excavation. Analytical studies and experimental investigation of such zones have been described by Talobre (14).

Photo-elastic studies were made which illustrated these effects for the joint pattern in the roof of the power station. Tests were also carried out with plaster of paris bars with joints in various directions, both under end forces and transverse load to examine the behavior both of smooth and rough joints. The most common cause of failure was by rotation and is illustrated in Fig. 9. The joint on the tensile side tends to open whilst the reaction between the two blocks on either side of the joint plane is transferred to a smaller area as the opening proceeds. Eventually, the joint fails by a shearing or crushing of the material at the point or points about which rotation is taking place. For two unconfined blocks failure took place almost in every case by shear of one form or another at this point. If the point of failure is confined, (c) Fig. 9, then although the initial fracture may be a shear fracture at an angle to the joint surface, the material is still capable of carrying load and is eventually crushed. With continued rotation the final failure was sudden, although ample warning had been given by the shear and crushing. This is one source of the "rock noise" mentioned later.

SUPPORT

In almost every underground job support of one form or another is used during construction to prevent collapse of weak zones in the rock. The conventional type of support is steel or timber "ribs" or "sets", using these terms in the wider sense to include ancillary support facilities.

During the last decade, rock bolts have come more and more into prominence as a means of supporting rock excavations. Prior to the World War II they had been used only to a limited extent. The view that rock bolts only "pin" or "nail" blocks or slabs of rock which are loose to the sounder rock behind them, is erroneous. Rock bolts are useful for this purpose and have been so used for a long time but, the use of rock bolts in this way, imagining that they are providing general support, can be dangerous.

It is well known from experience that there is a time factor involved in the failure of jointed rock - either complete failure leading to collapse of rock into the openings, or partial failure resulting in loosening of the surrounding rock. In the interval between the time of the blast which forms the opening, and the time when the rock mass either collapses or reaches stability, progressively increasing or decreasing amounts of movement occur along joints.

For either steel rib support or rock bolts it is extremely important for the support to be placed as soon as possible after blasting in order to prevent movement of the loosened rock. Once the rock has begun to move it is much more difficult to hold and, it generally requires a great deal more support.

On the T1 job both steel rib support and rock bolts were used. Probably the most spectacular use of steel ribs was in the excavation of the surge tank which is shown in Figure 10. In the large machine hall and transformer hall excavations it was decided to use rock bolts.

The manner in which bolts function in comparatively soft rocks, particularly those met with in coal mining, has been the subject of many investigations, but relatively little information was available concerning rock bolts in hard jointed rocks, such as granite and gneiss. The Snowy Mountains Authority therefore undertook a program of investigations to study how rock bolts function in hard, jointed rock and to establish techniques for their installation.

ROCK BOLT INVESTIGATIONS

The term rock-bolting, as used here, means the designed use of rock bolts to develop jointed rock into a structural entity which can competently play its part in a structure such as a power station.

It is important to distinguish between "anchor bars" and "rock bolts". Anchor bars are steel bars grouted into drilled holes in the rock and are not tensioned. Rock bolts are steel bars inserted in holes drilled in the rock, anchored at the end of the hole and tensioned by a nut and washer at the free surface of the rock. If they are grouted then the tensioning must be carried out before grouting. Rock bolts, due to the tensioning, create a principal compressive stress normal to the free surface of an excavation where, without them, there would be only principal stresses parallel to the surface. This is borne out by their very effective use to stabilize "popping" rock. This implies that, either at or immediately behind the free surface, the bolts form a diaphragm of material somewhat less in thickness than the length of the rock bolt, which can be used as a structural member whose properties can be ascertained and whose behavior can be assessed and designed for. It is obvious that, if rock bolts are to be designed to carry out the tasks enunciated above, then it is necessary to know their behavior in relation to both intact and jointed rock. This requires not only knowledge of the behavior of rock bolts but also of the behavior of jointed rock either with or without rock bolts.

Simple Joints and Rock Bolts

For underground excavations the rock is generally confined within the rock mass except at the free face. The case of loose surface blocks or slabs of rock offers no particular problems.

Individual rock bolts installed in jointed rock, in a direction such that the bolt makes an angle with the normal to the joint of less than the angle of friction, increase the friction on the joint by increasing the normal reaction across the joint. But more important, by holding the blocks together and preventing opening and rotation, they maintain the interlock of rough surfaces, and by this means bring into play the strength inherent in the interlocked rock. This effect can completely overshadow the friction effects. Therefore, it is important to establish what tension should be given to bolts when they are installed, and what the effects might be if this tension is released by rock movements or vibrations.

Theoretical analyses were made for a number of simple cases of blocks of rock at the surface of an excavation. These analyses are given in Appendix II. From these analyses it was concluded that (a) although rock bolts can exercise a very considerable stabilizing influence on a rock mass with relatively low tensions in the bolts it would be desirable in practice to maintain bolt tensions to some specified minimum. (b) If the tension in a bolt increases or decreases after it is installed it is a warning that relative movement of blocks of rock is taking place. (c) Initial movements of the jointed rock before placing bolts should be reduced to a minimum. (d) In a jointed rock, it rarely occurs that failure is due to sliding on the joints only; it is brought about by sliding and rotation. In either case sliding and rotation almost inevitably, in the case of confined rock blocks, cause the development of end forces on the block which have a stabilizing effect on the jointed material.

Machine Hall Roof

Figure 11 shows a model, made from lencite blocks which was constructed to show qualitatively the use of rock bolts to support the roof and abutment of an excavation similar to T1 Power Station. The blocks were cut in two directions, corresponding approximately with two main joint directions, A, B, Figures 6 and 23. The spacing, persistence, and direction are of course much more perfect and regular in the model than in the actual excavation.

The bolts were represented by thread and were tensioned by small steel springs. The pattern of bolting was similar to that used in T1 Power Station roof. The top and sides of the model could be loaded by springs to simulate conditions at depth around the excavation.

The behavior of the model was studied under various loadings on the top and sides. With the bolts in place and no loading on the top or sides, the model was quite stable, (a) Fig. 11.

Loading on the top of the model caused deformation of the whole mass by differential sliding along numerous joints in both directions and progressive removal of bolts caused blocks to fall out locally (b) Fig. 11 , and finally general collapse. Tension cracks tended to open between blocks below both abutments (Fig. 11). These developed quite differently on the right-hand side compared with the left-hand side, demonstrating the influence of the orientation of the joints in relation to the applied forces and the shape of the excavation. The bolts greatly restrained the opening of these tension cracks.

Photo-Elastic Investigations

A guide to the behavior of rock bolts and the stress condition they cause was obtained by photoelastic investigations. These covered such variables as tension in bolt, length of bolt, spacing between bolts, angle of bolts to free surface and the effect of bolts at re-entrant angles and other construction features. Examples from the series of tests with bolts normal to the free surface are given in Fig. 12. The models were CR. 39 plastic 1/4 in. thick and of varous widths and lengths to suit the matters being investigated. Bolts were simulated by loading heads connected by steel piano wire on either side of the plastic plate. Analyses of the stress patterns show very clearly that to develop a zone of uniform compression between the ends of the bolts, the ratio l/s should be not less than 2. Where l is the length of the bolt and s the distance between bolts. At this value the zone is relatively narrow whereas for l/s equals 3, it is approximately two-thirds of the bolt length (Fig. 12). Between the bolts, tension develops in the surface and there is a small area subject to tension which reaches its maximum mid-way between two bolts.

Similar investigations were carried out for bolts at 45° to the free surface with bolts in one direction and also crossed. In all cases, it was found that the l/s ratio had to be approximately 2 or more to give a zone of uniform compression. Fig. 12 shows stress patterns for an unconfined condition, i. e, no end restraint. Under this condition, uniform tension is induced in the material at right angles to the direction of the bolt. If end restraint or end loading is applied, then the tension is eliminated by a developed compression. Following the simple joint studies a model (Fig. 13) was constructed to confirm that material with regular joints could be stabilized by rock bolting. The regular joint pattern selected approximated the pattern encountered in the machine hall roof. The model consists of approximately 640 pieces of CR39, 1/4 inthick. Each block has smooth sides and stability is dependent entirely on the compression developed in the mass by the three rock bolts. End loading was not imposed but was developed against end restraint by tensioning the rock bolts. The model was free standing and the patterns shown in Fig. 13 were obtained with circularly polarized monochromatic light. It can be seen that stress concentrations occur at the corners of most of the small blocks, i. e., the blocks slide and rotate. The transfer of stress from bolt to bolt can also be seen together with comparatively large areas of fairly uniform stress. The conditions in this model were far worse than would be encountered in actual construction.

Another conclusion from the photoelastic investigations was that pattern bolting would be far more efficacious in controlling the unknown conditions behind the surface of an excavation than random spotting of individual bolts. Pattern bolting has been followed throughout the T1 Power Station excavation and the ratio l/s has been between 2 and 3 for this work. No jointing pattern in the rock is completely regular and the casual spacing of bolts based on superficial surface conditions could well lead to disastrous consequences.

Box and Bucket Models

Rock met with in practice varies from intact material to highly crushed and fractured rock. The photoelastic studies gave an insight into rock bolt behavior in an elastic, isotropic material. Therefore, investigations were made into the behavior of bolts when used to stabilize an uncoherent crushed rock mass like aggregate used in concrete. Most rocks encountered in construction lie between these two extremes.

The first model consisted simply of a rectangular box with a leucite front which was filled with small crushed rock 3/16 in. in size using model rock bolts, Fig. 14. The mass was compacted by vibration to eliminate some of the looseness inherent in this material. After tightening the bolts the box was inverted and it was found that this material did not fall out. A second qualitative test was made with an ordinary household bucket, Fig. 15. The lateral pressure developed by the bolts was sufficient to support not only the weight of the material in the bucket but an external load as well. It was concluded that the material was behaving in a quasi-plastic manner under the pressure created by the rock bolts, thus developing the lateral forces needed to support the rock mass and bolt.

A model of the machine hall roof using this material was also constructed (Fig. 16). The model had provision for side and top loading of the crushed rock material. After several trials, it was found that the material could be successfully bolted and could not be failed even although some hundreds of pounds was added as a top load.

Tension Zone

In the photoelastic investigations it was noted that at the free surface between rock bolts there existed a small tension zone. In the uncoherent crushed rock material there was a fall-out of loose material in this zone. Such fall-out occurred and can be seen in (b) Fig. 14, where small vaults have been created between the boundaries of the rock bolt washers. This was the first lead to an explanation as to why in practice relatively light steel wire mesh support between bolts was capable of stabilizing loose rock surfaces. The fall-out of this small amount of loose material in the tension zone between bolts can easily start an "unravelling" of the whole mass due to the movement of certain key fragments which, although carrying little or no load, may start a run if they are removed. This action was demonstrated with the machine hall roof model and examined by means of high speed moving pictures.

In the machine hall roof model the very small amount of support needed to cope with these tension zones was demonstrated by using one cigarette paper under each rock bolt washer. (Fig. 16). Following the completion of bolting the cigarette papers were burnt away and the whole mass still remained stable. All work on the small models was carried out without support between bolts. This undoubtedly led to a greater proportion of failures but it was done deliberately to examine the nature of the fall-out, the shape and extent of the vaults created, and the mechanism of failure.

Single Bolt Tests

Following the preliminary work, investigations were made using single bolts in cylindrical tubes, which gave experimental results in a form that could be analyzed more easily. These tests showed that when the bolted anchored rock was just stable there appeared to be a relationship between the clear space s between bolt washers, the mean particle size m, the bolt length l and the bolt tension B. Although generalized forms of the relationship have not yet been fully resolved the ratio

$$F = \frac{s}{m}$$

proved to be a useful parameter. For example, in investigating the effect of particle shape, a series of tests were made using marbles in a vertical box having a width slightly greater than the marble diameter and fitted with an adjustable opening in the bottom. For an F number of 3 the marbles were always stable. If the F number was 4 they invariably failed. At present this parameter is not sufficiently established to be used for design purposes.

To further illustrate both behaviors, another model shown in Fig. 17 was also constructed. It consisted of 3 in. x 1/4 in. cylindrical rods of polystyrene made up into the form of a beam. The circular shape was selected as probably being the least stable which could be used. The F number adopted was 3 and the bolts were spring loaded so that bolt tension could be determined. The loading jig enabled end loads to be applied or measured and also provided for free or fixed end conditions. The fall-out between bolt washers, thus creating vaults, can be easily seen. The beam was loaded with a concentrated load (b) Fig. 17 and was found to behave approximately elastically up to failure, which, when it occurred, was sudden and catastrophic. For an F value of 4 the beam was always unstable.

Crushed Rock Tests

Following the small-scale tests larger scale tests for material similar to that which might be encountered in excavation practice and using bolts of large size were begun and are continuing. The testing apparatus consists of a box 4 ft. x 4 ft. x 4 ft. (Fig. 19) with vibrating wire pressure cells mounted in the sides to measure lateral pressures, and a hydraulic jack assembly to apply and measure central loads.

From the small scale tests it was concluded that bolting of the crushed rock was creating from the uncoherent fragmented rock mass a quasi-elastic diaphragm between the ends of the bolts. Therefore, in these larger scale tests it was decided to construct diaphragms with various thicknesses. The method of using the testing rig was to bolt a temporary floor of planks underneath the box, place the rock bolts in position, fill the box to the required depth, assemble and tension the bolts, and then remove the floor. Following this the diaphragm was loaded, and deflections, loads, lateral pressures and bolt tensions measured.

The results of several of these tests are shown in Figs. 18 and 19, and in Table 2. Results of typical tests with material, 1.5 to 2.25 in. in size with an F number of 4.25, and a bolt length of 23 in. are given by curve 1, Fig. 19. After removal of the bottom the loose rock in the tension zone between the washers fell out, creating the vaults. Load was applied and although there had been no indications of failure, a few fragments dropped out at 7,000 lb. (D Fig. 19) and commenced an unravelling leading to failure. In order to prevent this and to demonstrate how flimsy may be the support needed between the bolts, the test was repeated with 2 in. by 24 gauge chicken wire netting placed beneath the bolt washers, (a) Fig. 18, but not attached to the sides of the box. There was little bulging of the wire and the model was loaded and unloaded in accordance with the sequence shown graphically in Fig. 19. Cyclic loading was applied at various loads, and small permanent deflections occurred with each cyclic loading in the early stages, e.g. between 3 and 4 in., Fig 19. At 5, 7, 9, Fig. 19, it will be observed that, after a number of cycles, further permanent deflection ceased and constant hysteresis loops were obtained. At 8, the load was left on overnight and there was a relaxation of strain and load in the 12 hours which illustrates the time effect noted in practice. After bringing it back to the maximum load, viz. 13,000 lb., the load was reduced to zero and a series of cycles at 0 to 10,000 lb. applied. At this stage, 10, Fig. 19 the chicken-wire was cut away, (b) Fig. 18 when fall-out in the tension zones occurred with typical vaults appearing between the bolts. It was again loaded from 10 to 11 during which rock fragments were falling out, and left standing at 13,000 lb. for an hour. Occasional fragments fell, giving a small relaxation in strain and load. The mass was then saturated with water and it failed by general collapse.

Significant features illustrated in Table 2 and Fig. 19 are

(a) the general curve of the envelope 2, 3, 4, 6, 8, 11 is typical of a brittle material,

(b) the unloading and reloading curves for the cyclic loading follow a power law of the form W equals b $(h-a)^n$ and show elastic hysteresis,

(c) the crushed rock material behaves quasi-plastically and after working has a "yield" point which increases in a manner analogous to "strain hardening" for ductile metals.

The other test results given in Table 2 are for crushed rock identical with that used for tests in Fig. 26, with 4 bolts instead of 9. However, the bolt tensions were increased from 5,000 lb. to 15,000 lb., following results from the small scale tests which showed that if bolt tensions were increased, then F numbers could be increased also. 2 in. by 24 gauge chicken-wire was used under the bolt washers. It was anticipated that the wire in this case would be carrying a larger load than in the earlier test and this proved to be the case. Loading was carried through a series of cycles and the material was quite stable up to the maximum load of 13,000 lb. Again, the chicken-wire was cut but this time failure occurred by unravelling when about three-quarters of the chicken-wire had been removed.

<u>These investigations demonstrate</u> -

(a) that crushed rock can be stabilized by rock bolts and the diaphragm formed between the ends of the bolts behaves as an integral, quasi-plastic structural member,

(b) that loading and unloading causes "working" of the fragments with reduction in tension in the bolts thus demonstrating the need for retightening bolts after vibration or other working,

(c) the very flimsy nature of the support that is needed to cope with loose material in the tension zone vault between the bolt washers,

(d) the importance of supporting such loose material so as to prevent unravelling of the whole fractured mass behind the bolts when under load.

Bolts have been used in such locations at several points in the T1 Power Station work. Generally speaking steel mesh lagging was not needed in the T1 Power Station except over the upstream wall, Fig 7 , where it was placed to prevent danger to workmen from small loose rock fragments rather than for structural reasons.

It should be stressed that analyses of the conditions pertaining in the bolted crushed-rock diaphragm have not reached the stage where a tried and working formula can be put forward for routine use.

STRENGTH AND BEHAVIOR OF BOLTS

At the time these works began, much information had been published about slot and wedge type bolts in sedimentary rocks, but very little information was available on the use of these bolts in hard rock. The opinion seemed to be held, almost universally, that the slot and wedge type anchorage was quite satisfactory in the softer rocks, but was not satisfactory in hard rocks such as granite. Therefore tests were carried out to ascertain the competency of bolts in granite and to determine a satisfactory technique for installation. The testing program centred around the slot and wedge type anchorage, and 234 bolts of this type were tested. Some 60 bolts with expansion type anchorages were tested for comparison purposes. Bolts of various types are shown in Figs 28 and 29.

The rock bolts used in T1 consist of a mild steel bar, 1 in. nominal diameter, with 6 in. of rolled thread (1 in. Whitworth) at one end and, at the other end, a flame-cut diametrical slot, in which the flame-cut wedge was inserted. A dimensioned sketch of the bolts is given in (a) Fig. 28 . The diameters of the drill holes were maintained between 1-1/4 in. and 1-3/8 in. and lengths were 10 ft. 15 ft. and 20 ft. with occasional odd sizes for special work.

The ultimate strength of the bolts was approximately 40,000 lbs. (60,000 psi) and the yield point was 22-24,000 lbs. (33-36,000 psi).

The so-called "strength" of a rock bolt is determined by its anchorage and a bolt is considered to have failed if it is impossible to hold the required tension in the bolt owing to continuing slip of the anchorage. In practice, tension in the bolt would normally not be greater than the yield point of the steel and therefore the breaking of the steel shank does not arise. In the tests a number of bolts were taken well over the yield point of the steel and bolts were broken either in the thread or in the prongs in the vicinity of the wedge. The load in the bolt at which the anchorage slips depends on the driving technique, the relative magnitudes of hole diameter, bolt diameter, slot width, and wedge dimensions.

The relative dimensions of the anchorage and the installation technique are satisfactory and the bolt can be considered competent if the tension in the bolt can be raised to the yield point of the steel or other required "proof" load without significant anchorage slip.

The test procedure consisted of drilling the hole, inserting and driving the bolt, pulling the bolt with a hydraulic jack assembly and measuring bolt tension, anchorage movement, and bolt elongation. The tests were all carried out with 1 in. nominal diameter bolts and included drill hole diameters from 1-1/16 to 1-5/8 in. wedge sizes from 1/2 to 1 in. and bolt lengths from 2 to 20 ft. Driving time, air pressure, length of bolt, and the number of impacts per minute by the driving hammer, were also varied and the effect on anchorage competency noted. It was found that no trouble was experienced in obtaining a satisfactory anchorage if the air pressure at the driving hammer were not less than 75 psi, (preferably 85-100 psi) the driving time was 20-30 seconds, and the hammer gave between 2-3,000 impacts per minute.

Variation in anchorage efficiency was observed between holes which were drilled vertically downwards, horizontally, and vertically upwards. The vertical-up holes were self-cleansing, and after cleaning drilling sludge from horizontal and vertical-down holes, it was found that these holes gave results equal to the vertical-up holes.

Although it may be reasonable to assume that the longer bolts would absorb more energy and therefore the anchorage would not be driven as satisfactorily as it would with shorter bolts, the test results, covering bolt lengths from 2 to 20 ft. were not decisive in this regard. However, work in the power station did show that more care and attention were required in driving the 20 ft. bolts than the shorter bolts if satisfactory anchorages were to be obtained.

The various dimensions used in the anchorage assembly were correlated by

$$\epsilon_B = \frac{w + d - t - D}{D} \tag{7}$$

in which D is diameter of drill hole, d is diameter of bolt, t is width of slot, w is wedge thickness, and ϵ_B is the diametral strain of the anchorage assembly assuming the assembly is deformed to the hole diameter.

The test results in terms of ϵ_B and two "proof" loads are summarized in Table 4. This shows that, provided ϵ_B is 0.15 or greater, there should be no difficulty in achieving a 95 per cent satisfactory performance for bolt loads up to 20,000 lbs. and 80 per cent for bolt loads up to 30,000 lbs. If ϵ_B is less than 0.15 then only 65 per cent to 46 per cent of the bolts are satisfactory.

Anchorage Behavior

When the wedge is inserted in the slot and the wedge and bolt driven home against the end of the drill hole, the prongs of the bolt spread, touch the sides of the drill hole and are either plastically deformed by the rock or plough a groove into the rock or both. If the prongs of the bolt do not plough a groove in the rock and the wedge is driven home, then ϵ_B the diametral strain of the anchorage assembly is given by Equation (7).

In the course of the tests it was found that, on initially tightening the bolt after driving, anchorage movements took place. As the loads increased further movements sometimes took place. When the anchorage did not slip without further increase in load, the load could be removed and again applied to its previous value before further slip took place. In other words, the anchorage itself had a "yield" point and behaved as though the junction between the steel and the rock had the property of "strain hardening".

A number of anchorages were removed from the rocks,and it was observed that in the center of the contact area rock fragments and minerals were adhering to steel, and that in the central area of contact the steel had been severly deformed plastically and showed characteristic shear slip surfaces. The minerals adhering to it also showed shear slip surfaces. These minerals appeared to have also been pulverized and compressed under very high pressure. That the steel in this area had undergone very severe plastic deformation was confirmed by a hardness survey of the contact area. The unworked bolt shank had a Brinell hardness number of 150 compared to 160 just inside the edge of the contact area and a maximum of 250 in the center of the contact areas.

Severe plastic deformation as was observed begins at a pressure equivalent to about 0.4 of the Brinell hardness number (10), (11). With a Brinell value of 150 kgm per sq. mm. (214,000 psi) the pressure for plastic deformation would then be about 85,000 psi.

At the centre of the contact areas both the steel and the rock are under a condition of high triaxial compression and are entering the range where it might be expected that the rock minerals would also be deforming plastically (8), (9). Under these conditions there is probably strong adhesion between the steel and the rock as well as a very intimate and strong mechanical bond at the junction.

These features of the steel-rock junction are consistent with the stick-slip motion of the anchorage under increasing load which was noted during the tests.

The deformation of the anchorage is elastic as well as plastic and if the anchorage "sticks" at any applied load then this load must be exceeded before it "slips", i.e., the "stick-slip" load increases as the elastic yield point of the steel is increased by strain hardening. The elastic lateral deformation of the anchorage assembly under increasing bolt tension in accordance with Poisson's ratio will also contribute to the "stick-slip" nature of the anchorage movement.

In hard rock, bolts with a slot and wedge anchorage can be installed satisfactorily and, if so desired, can be given an intial "proof" load to ensure a margin over the working load.

Torque and Bolt Tension

The most convenient method of obtaining a required tension in a bolt is to use a torque-controlled impact wrench.

Tests were made to determine the torque-tension relationship for the bolts used in T1. The tests gave a torque-tension relationship in accordance with the equation

$$T = kBd \tag{8}$$

where T is the torque applied to the bolt nut in pounds-feet, k is a constant, B is tension in the bolt in pounds, and d is bolt diameter in inches. The average value for k was 0.0166.

Field conditions, particularly angle between the bolt axis and the plane of the bearing plate, can cause large deviations from **Equation (8)** and care is needed to ensure that required bolt tensions are obtained. Tapered washers as illustrated in 28, Fig 30, were used in T1 to obtain these effects.

ROCK BOLTING PRACTICE IN T1

During the excavation of the T1 Power Station which has been described elsewhere (4), rock bolts were used to create from the jointed rock in the roof a self-supporting structure, pending the installation of the permanent concrete ribs. In the walls of the machine and transformer halls, rock bolts were used for a similar purpose, but were grouted for permanency. Brief details of the number of bolts used and their locations are given in Table 3. A typical bolting pattern for the machine hall is shown in Fig. 20. The bolts consisted of mild steel bars 1 inch in diameter, mostly 10 or 15 feet long, with a slot-and-wedge type anchor and furnished with 6-inch or 8-inch square steel plates for bearing against the rock surface. During installation they were stressed to a nominal load of 15,000 pounds tension.

At the commencement of this work, the workmen were apprehensive that rock bolts would not be adequate protection in an excavation of this size. This feeling culminated in a request by the Contractor to permit the use of light, supplementary steel support in the machine hall excavation, Fig. 21. Although the supplementary steel support, which consisted of 8 in. x 6 in. I-beam sections and 12 in. x 8 in. I-beams spanning the pilot tunnel, would only take a light rock load, provision was made to install gauging points on a number of the steel columns, so as to ascertain if it was carrying loads. Deflection measurements on the 12 in. x 8 in. I-beams were also made. In the vicinity of Rib No. 16, (Fig. 21), where the geologists had indicated that the intersection of joints A and B, (Fig. 21 and Fig 23), could give trouble, the gauges on one of the steel columns on the downstream side showed that it was carrying a load of approximately 100 tons. More rock bolts were installed in this area and particular attention given to any unusual indications of rock noise. Later, when this particular steel support was removed in order to enable placing of the permanent concrete ribs, no fall of rock occurred and no rock movement could be detected. Similarly, all the other steel support was removed with no trouble apart from minor scaling. Doubtless the steel rib support had a good psychological effect and prevented small pieces of rock falling from the roof.

Following the experience in the machine hall and improvement in the technique for installation of bolts, the transformer hall was excavated without difficulty and supported using rock bolts above. (Fig. 22).

The bolting in both excavations was carried out systematically to a pattern. Bolts were usually 10 or 15 ft long and spaced 4 or 5 ft apart, so that the ratio of length to spacing varied from 2 to 3. Within this pattern the location of individual bolts could be varied slightly to take account of the jointing in the surface layer of rock, bolts being placed towards the centre of joint blocks rather than on their edges. Where there were well-defined persistent joints or minor faults the pattern was modified. This was done, for example, in the case of the intersection of fault A and fault B and associated joints (Fig. 23).

In T1 it was found that about 5 per cent of bolts became loose, i.e., lost some of their tension, up to 10 to 15 ft. behind the working face. The majority of loose bolts were caused by small pieces of rock becoming loose immediately under the bearing washer. However, most bolts were re-tightened satisfactorily by application of the impact wrench or by inserting extra packing between the rock and the nut. In the machine hall all wall bolts which could be placed on a slight downward slope were grouted so as to provide reasonable protection against corrosion and ensure their continuity as a means of support.

Rubber pressure pads were used under a number of selected bolts to determine any variation of tension in the bolts with time. These pads were calibrated and could be read fairly easily. They gave an indication of the relative behavior of bolts, indicating any zone where bolt tensions were decreasing or increasing, and thus gave warning of movement or relaxation in the rock before dangerous conditions could develop. Such indications were always qualified by the results from the rock noise stations and the other behavior instruments. In installing the bolts it was standard practice to check the torque on every bolt immediately following installation. If it fell below the specified torque, then the bolt had to be re-tightened or, if still unsatisfactory, re-driven or replaced. The torque on all bolts recently installed was periodically checked. In addition, pull tests on selected bolts were made from time to time by means of hydraulic jack testing equipment to ensure that the required standards for installation were being maintained.

EXCAVATION BEHAVIOR

The behavior of a rock structure such as the T1 Power Station is of great importance, both during and after construction. Not only do the results of measurements of movement give a check on design assumptions, but also progressively monitor the construction work so that unusual or hazardous conditions can be anticipated, and preparations made to cope with them.

In T1 the following measurements were taken to enable assessment to be made of performance and behavior:

(1) Measurement of strain in the reinforced concrete arch ribs by means of electric resistance type strain meters embedded in the concrete, and Huggenburger deformeter points fixed to the sides of some ribs.

(2) Measurement of rock and concrete movements during construction by means of sensitive bubble clinometers and by precise survey methods.

The strain gauges were located in selected concrete ribs at three points shown in Fig. 25 with connection to a central reading location at one end of the machine hall. Strain measurements were taken at least once per week and these were plotted as graphs and distributed to construction, design, and scientific service personnel for information and analysis.

Instruments to indicate movement of the rock walls and of the concrete abutment beams were sensitive clinometers reading to 2 seconds of arc. Base plates for these instruments were installed at four positions on the upstream abutment beam, five positions on the downstream abutment beam, six positions on the upstream wall, and four positions on the downstream wall of the machine hall excavation. Although these clinometers only showed angular rotations, they were used in conjunction with other information to assess and interpret the movements that took place.

Targets for precise survey work were installed on the abutment beams and on the walls of the machine hall. Movements of these targets as small as 0.01 in. could be observed.

With the installation of the machine hall roof ribs (a, Fig 24) the first stage of the excavation was completed and, from this time, viz. February, 1957, readings were taken of the strain recorded by the strain meters in the roof ribs.

In the early part of May, 1957, there was a sudden increase in the amount of strain being recorded. This coincided with the excavation of the machine hall reaching a level 16 ft. below that at which the concrete ribs were placed (b, Fig. 24). At the same time, a number of the clinometers showed rotation, those on the abutment beams showing the most rotation and in a direction away from the machine hall centre line. The results of these readings were watched closely and the strain in the ribs where the excavation was proceeding most rapidly, continued to show a rapid increase to about the end of June, 1957, when the excavation was some 45 feet below the abutment level. A typical example (Rib 9) is given in Fig 25.

Estimates of the stresses in the roof ribs on account of this strain were made with allowance for temperature effects and creep, and indicated that stresses of the order of 1,500 psi were probable.

In order to provide a basis for forecasting the trend of these movements, photo-elastic laboratory investigations were undertaken to elucidate the general mechanism causing the strain. The procedure was to cut a model with the initial excavation of the elliptical roof section, place it under strain in the polariscope and then insert a model roof rib, thus copying the construction procedure. The model was then progressively cut in the sections I, II, III, and V, (Fig. 25). Models were tested in vertical, horizontal and 45° stress fields. Qualitatively, the model results showed the same behavior as the strain readings in the power station ribs. (Fig. 25). These results also indicated that the strain would continue to increase, but at a slower rate as excavation proceeded.

With the increase in strain and stress in the ribs, together with the rotation of the abutment beams indicated by the clinometers, it was anticipated that there may be cracking or spalling of the concrete at points of stress concentration where the ribs join the abutment beams. This did take place and by July 1957 cracks and surface spalling had occurred on the inside edge of Ribs Nos. 11-15, where the rib joined the abutment beam on the downstream side of the machine hall. Also there were several cracks in the abutment beam (Fig. 26). From concrete tests made during construction, the strength of these ribs at the time when cracking was observed was of the order of 6,000 psi which was well above the estimated average stress in the ribs, and was consistent with the rotation of this abutment, shown by the clinometers, causing stress concentration several times greater than the average stress in the concrete rib.

The results of the photo-elastic analysis and the measurements in the power station indicate that, as the excavation proceeded downwards, the walls of the power station deformed in a manner qualitatively indicated by the dotted lines in Fig. 27, i.e. at the abutment level, they were moving upwards and inwards towards the center line. These movements were consistent with stress relief in the walls following excavation particularly in view of the high horizontal stress originally in the rock.

Although readings on the precise survey points were difficult to obtain owing to the construction conditions, and considerable errors could be anticipated owing to refraction from air temperature gradients, the trend of the readings confirmed the above hypothesis. During this period, particular attention was given to indications of any unusual rock noise. The rock noise surveys showed nothing unusual, thus indicating that there was no fracture, shearing or slipping on rock joints taking place. A careful examination of the walls of the power station was also made by the geologists and, apart from minor local occurrences which could be associated with the progress of the excavation, no movements or new cracks were found. As anticipated these movements continued while excavation was in progress and also for some time afterwards. A typical example of the total movements up to March 1958, when progressive movement had practically ceased is shown in Figure 27. One factor contributing to the asymmetric nature of the strain in the ribs and the larger movement of the downstream wall was probably the minor fault B, (see Fig. 27) which has been described by Moye (5). He also has described the rock noise measurements carried out for this work.

The strain meter readings indicating high loads in a number of the roof ribs, which were confirmed by the cracking and spalling of the concrete in Ribs Nos. 11 - 15 on the downstream side of the machine hall and the clinometer readings, would normally lead one to expect fracture and cracking of the rock and opening up of joints in the walls of the machine hall excavation, particularly just below abutment level, where maximum movement and rotation has taken place. That such symptoms did not appear was due to the rock bolting, the general pattern of which is shown in Fig. 20, creating a diaphragm (some 7-10 ft. thick) around the whole excavation which behaved as an integral elastic shell. This conclusion is consistent with the results of the investigations into the action and behavior of rock bolts.

CONCLUSION

In order that underground works can be designed and constructed with confidence and efficiency, much investigation and research is needed. A few examples are:

1. In view of the basic major influence which the natural state of stress in a rock mass has on the behavior of the rock around underground openings, a reliable method of determining the actual stresses at the underground site is urgently needed. The instruments used should be rugged to give reliable results under difficult field conditions, and preferably be capable of operation at depth in a diamond core drill hole. A corollary to this is to correlate the natural state of stress to the geology of the site, and its depth and the surrounding topography.

2. The development of a "direction" geophone for use in sub-audible rock noise measurements so that the center of disturbance, i.e., the location of the rock movement causing the noise, can be accurately located.

3. A method for determining the depth of penetration into the rock of blasting effects.

4. Investigation of the behavior of jointed rock in situ and the properties of rough interlocked joints under various confining forces.

5. Investigation of the behavior of rock bolts in stabilizing and reconstituting jointed rock of various types and the extent to which bolted rock can be used as part of the permanent structure of underground works. This implies the development of a satisfactory and permanent method of protecting rock bolts by cement grouting or other means.

6. Comprehensive investigation of the strength and behavior of various types of rock bolt anchorages in different kinds of rock to provide urgently needed basic data for the design and construction of rock bolted structures.

7. Investigation of the behavior and strength of steel rib support and its use under various underground conditions.

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TABLE 1

AVERAGE ROCK PROPERTIES

ITEM	Granite (Type 1) Exploratory Tunnel	Granite (Type 1) Machine Hall	Granitic Gneiss (Type 2) Exploratory Tunnel	Granitic Gneiss (Type 2) Machine Hall	Granitic Gneiss (Type 2) Transformer Hall
Specific Gravity (Bulk)	2.73	2.71	2.70	2.73	2.71
Porosity (% Volume)	0.30	0.70	0.30	0.30	0.50
Compression					
Ultimate Compressive Strength (1000 psi)	19.5	21.0	21.0	19.5	14.3
Limit of Proportionality (1000 psi)	13.2	12.9	12.5	12.5	11.6
E (10^6 psi)	10.3	8.4	10.2	9.4	7.3
Poisson's Ratio ν	-	0.22	0.25	0.25	0.23
Angle of Fracture	65°	67°	70°	73°	70°
Tension					
Ult. Tensile Strength (1000 psi)	1.18	1.11	0.93	1.10	0.99
Limit of Proportionality (1000 psi)	0.88	0.58	0.83	0.56	0.40
E (10^6psi)	7.6	6.5	9.3	7.3	6.7
Angle of Fracture	5°-10°	5°-10°	5°-10°	5°-10°	5°-10°

ITEM	Transformer Hall Granitic Gneiss (Type 2)			Machine Hall Granitic Gneiss (Type 2)		
Triaxial Tests						
Confining Pressure 1000 psi	0	5	15	0	5	15
Axial Ult. Compressive Strength 1000 psi	14.3	35.3	60.2	19.5	49.9	80.3
Angle of Fracture	65°	60°	65°	65°	65°	60°

T A B L E 2.

ROCK BOLT TESTS - CRUSHED ROCK.

SELECTED RESULTS

Test	ROCK SIZE		Bolt	SPACING		Washer	F=	Bolt	Lateral	Load		
	Range ins.	Mean ins.	Length l in	Bolts s_1 ins.	Clear s_2 ins.	Size ins.	$\frac{s_2}{m}$	Tension B 1,000 lb.	Pressure p lbs./ sq.ft.	W 1,000 lbs.	Deflection h ins.	Remarks
1	3-5	4	46.	23	13	10x10	3.25	7.50	821	-	-	
4								2.00	998	13.0	1.600	Failed by reducing
Bolts								1.35	990	10.0	1.908	B to zero
2	1.5-2.25	1.875	23	16	8	8x8	4.27	5.00	1750	-	-	2 in. x 24g Wire netti
												support
								5.00	2835	13.0	0.546	
								4.85	2350	0.0	0.415	Wire netting removed
9												
Bolts								4.85	2575	13.0	0.619	Failed after saturatio
3	1.5-2.25	1.875	39	16	13	10x10	6.93	15.00	1520	-	-	2 in x 24g Wire nettin
4								10.40	2440	13.0	0.825	support
Bolts								10.40	2140	13.0	0.983	After 35 additional cycles
												Failed on removing wir support.

Average Weight of Crushed Rock = 105 lbs. / cubic foot

TABLE 3

ROCK BOLTING STATISTICS

Feature	Number of Bolts			Total Length	Area Sup-ported	Area per Bolt for Pattern Bolting	Length of Bolts Used
	Plain No.	Grouted No.	Total No.	1,000 ft.	1,000 sq.ft.	sq. ft.	
Machine Hall							
a. Roof	2,194	-	2,194	26.7	25.4	11.6	) 10', 15', 20') Few 7' and) 12'
b. Walls	261	861	1,122	16.5	30.0	27.0	
Transformer Hall							
a. Roof	1,366	-	1,366	17.0	10.0	6.6	)) 10' and 15')
b. Wall	40	83	123	1.4	6.0	44.0	
Tailrace Portal - Open Cut	62	177	239	2.4	15.0	63.0	10'
Totals	3,923	1,121	5,044	64.0	86.4	17.1	
Tunnels and Shafts	940	-	940	Not pattern bolting			8' and 10'
TOTAL	4,863	1,121	5,984	-	-	-	

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TABLE 4

SLOT AND WEDGE ANCHORAGE TESTS

1 in. Dia. M.S. Rock Bolts

ϵ_B	< 0.15		0.15-0.25		0.25-0.35		> 0.35	
	Proof Load - 20,000 lbs.							
	No.	%	No.	%	No.	%	No.	%
Total Bolts Tested	31	100	98	100	68	100	18	100
Competent	20	65	93	95	66	97	18	100
Failed	11	35	5	5	2	3	0	0
	Proof Load - 30,000 lbs.							
	No.	%	No.	%	No.	%	No.	%
Total Bolts Tested	22	100	38	100	20	100	4	100
Competent	10	46	31	82	17	85	4	100
Failed	12	54	7	18	3	15	0	0

FIGURES

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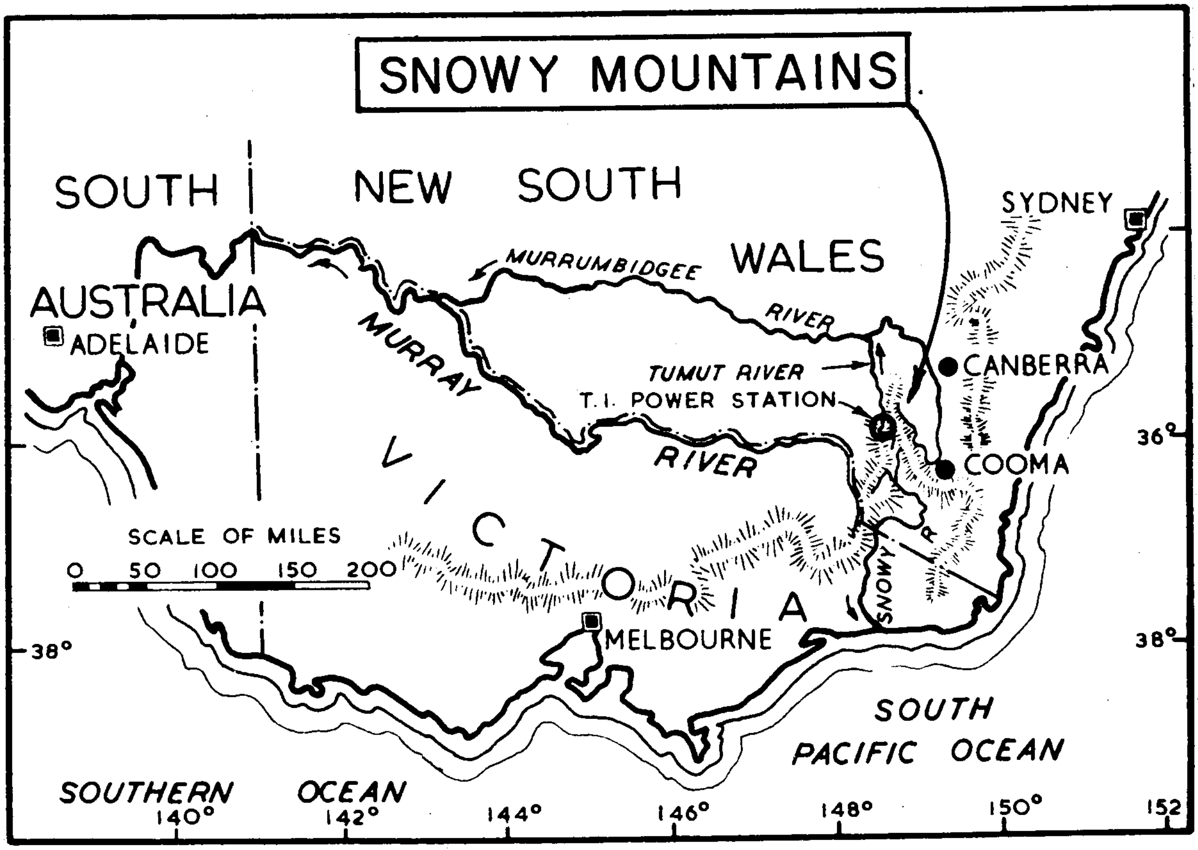

Fig. 1 — Location map

Fig. 2 — Tumut valley

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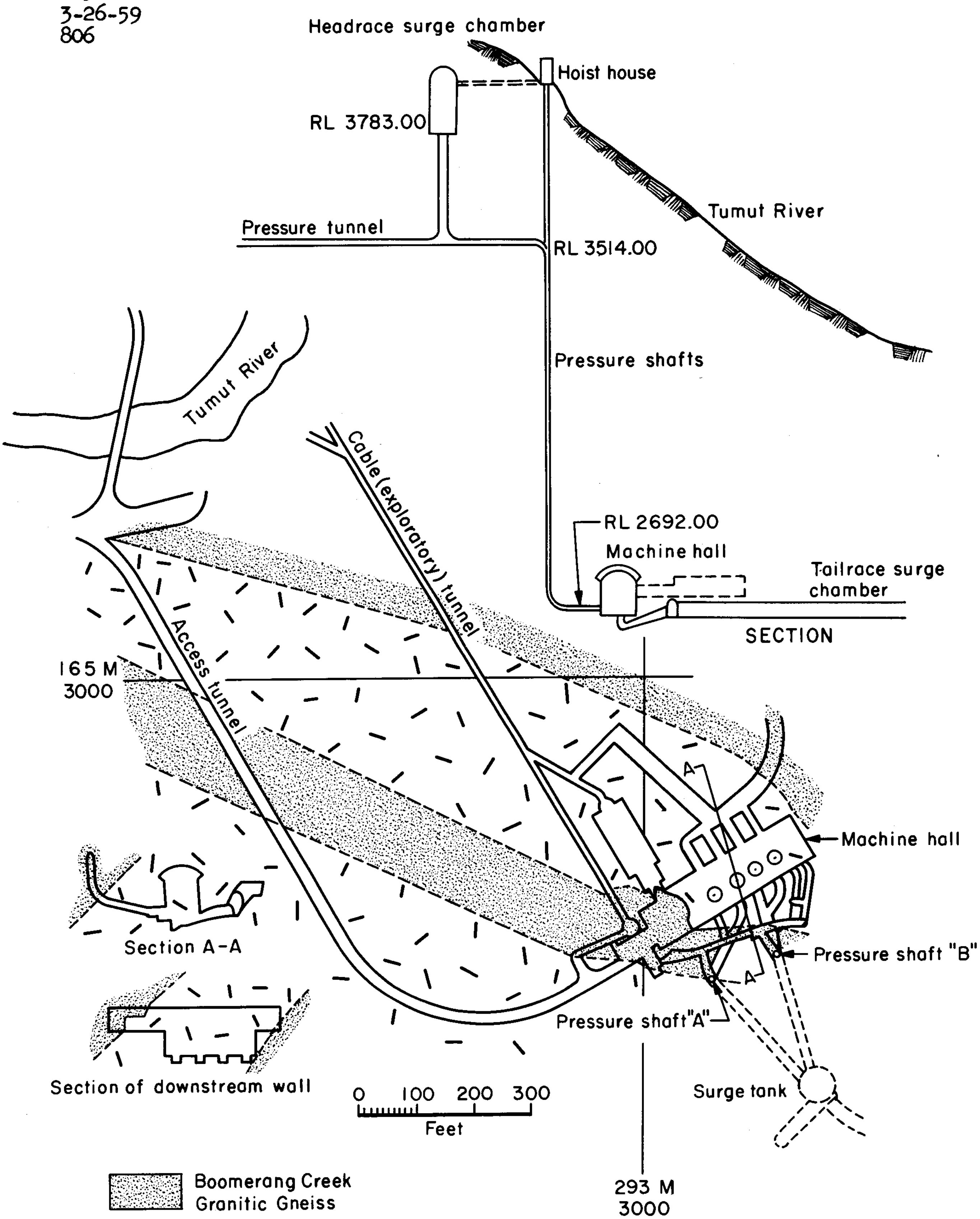

Fig. 3 — Plan showing rock types T. I. power stations — general layout

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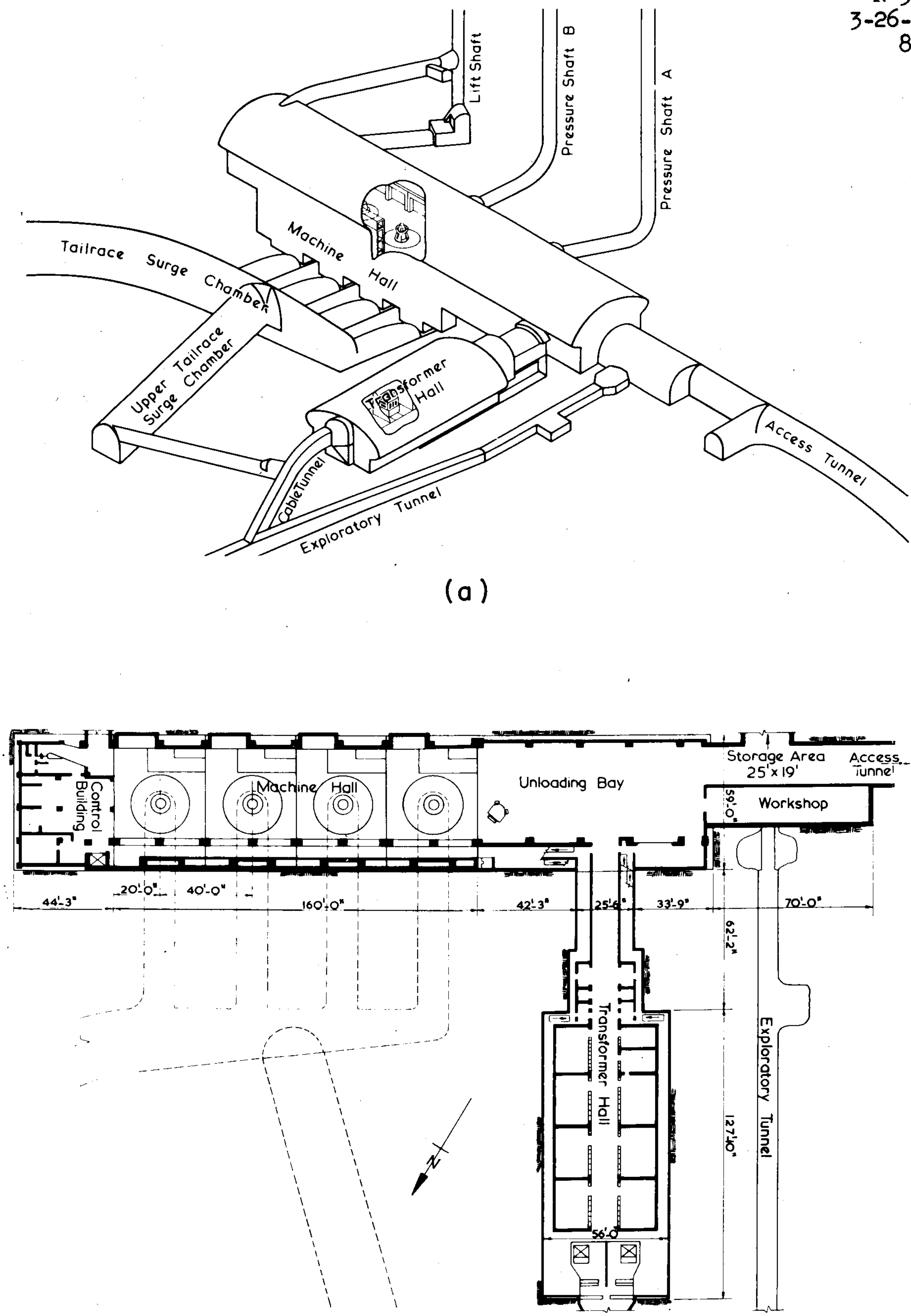

Fig. 4—General arrangement of T.'I. power station

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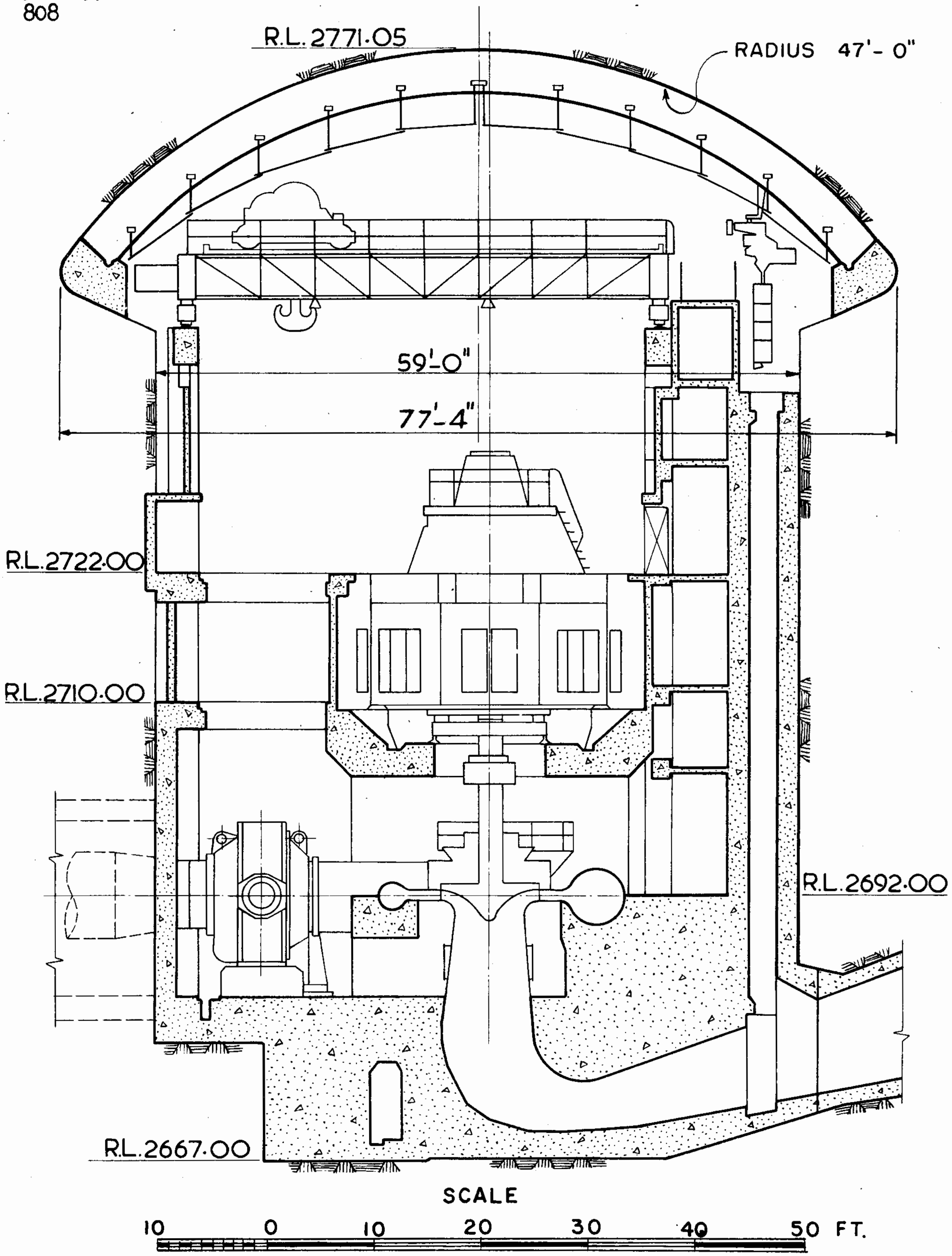


Fig. 5 —— Cross section of machine hall

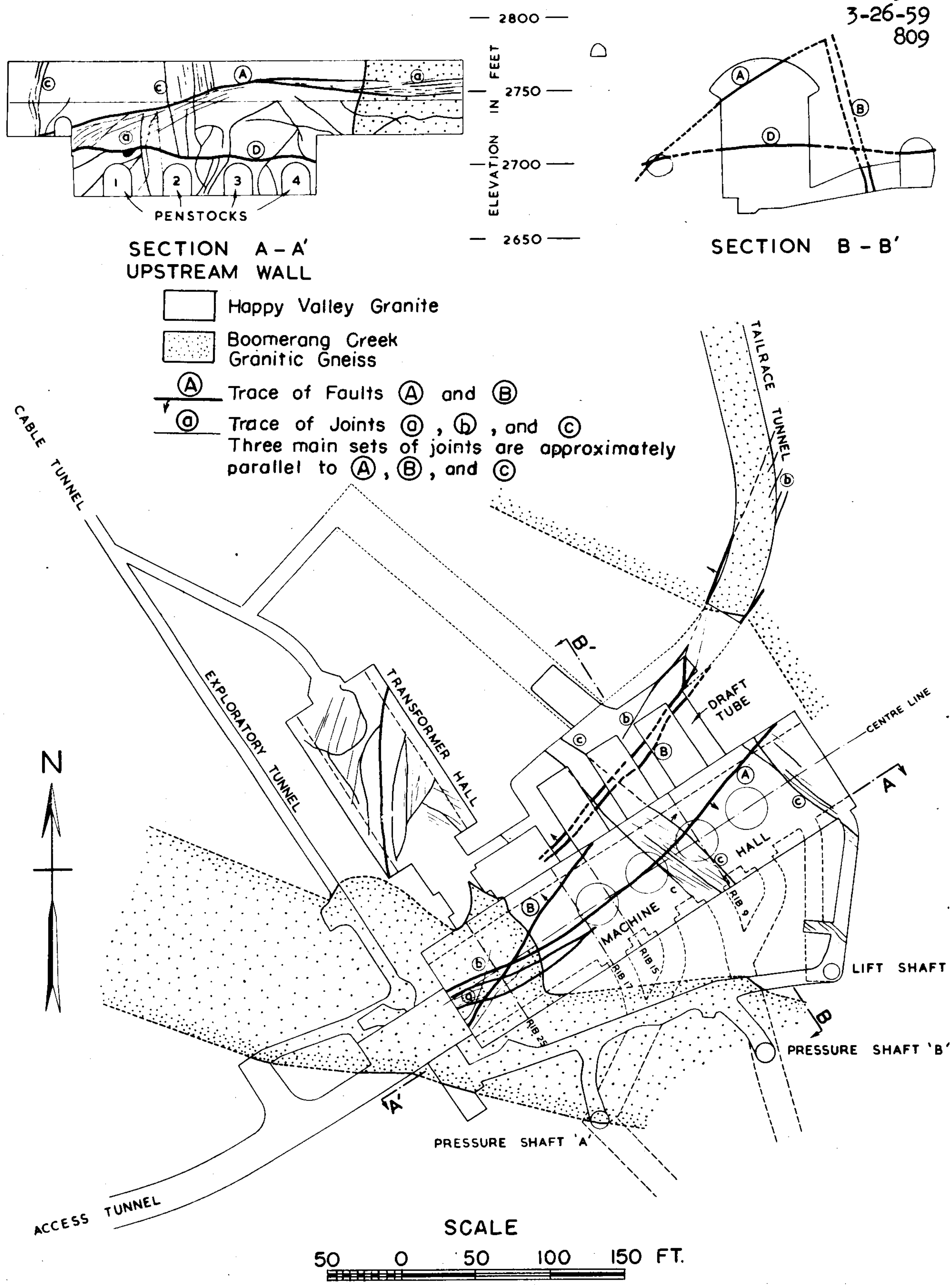


Fig. 6 —— Geology of site

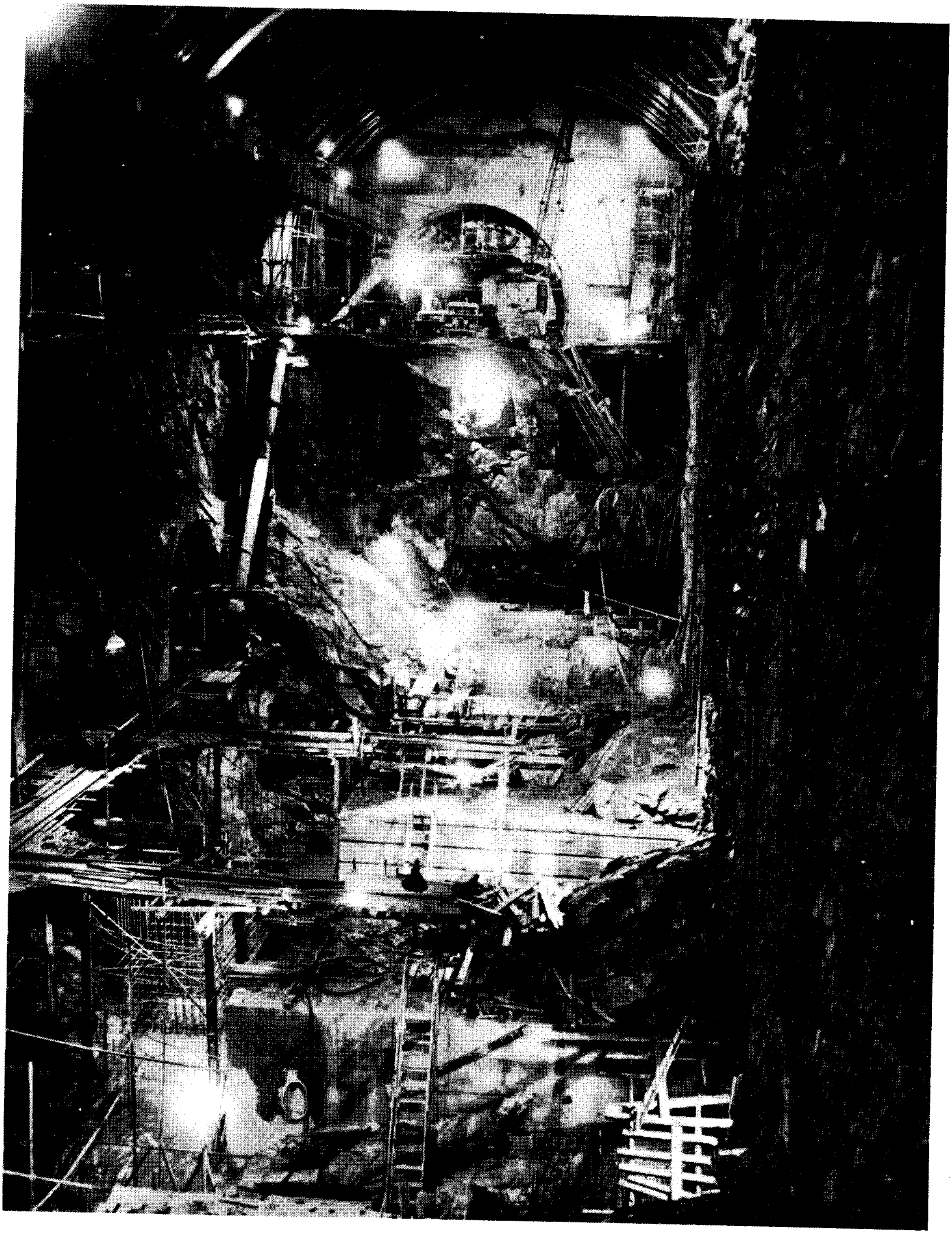
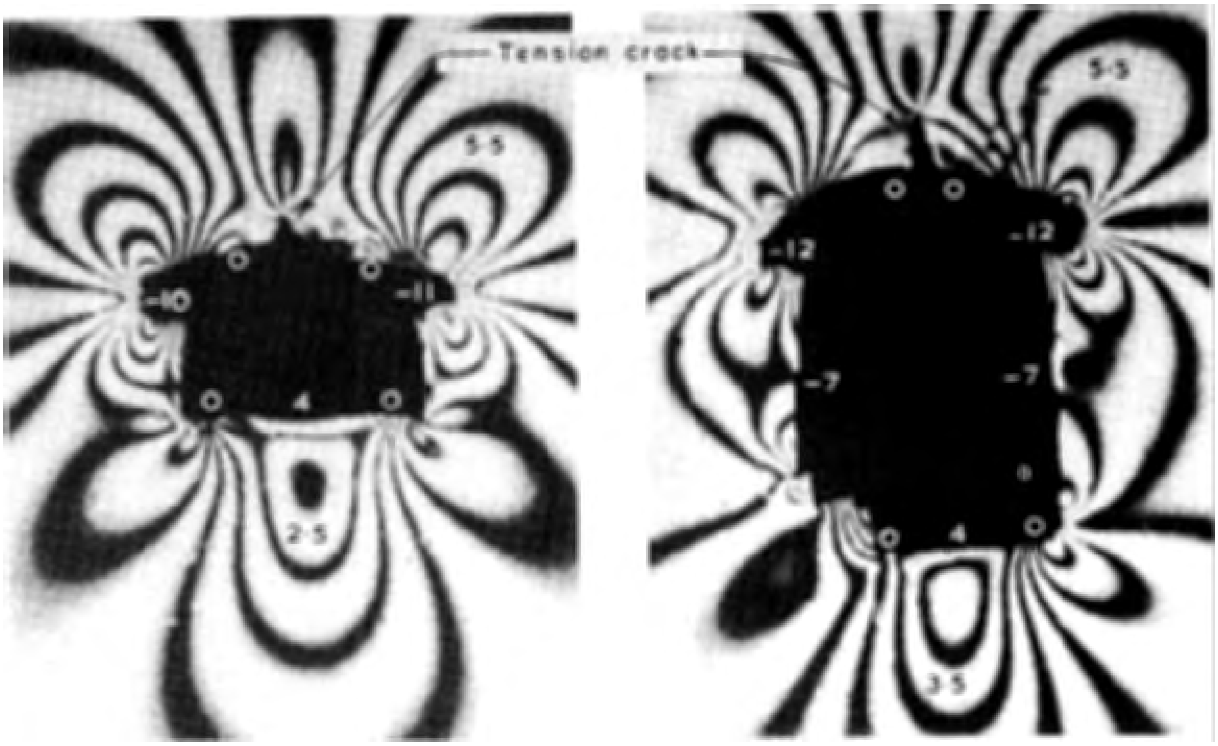
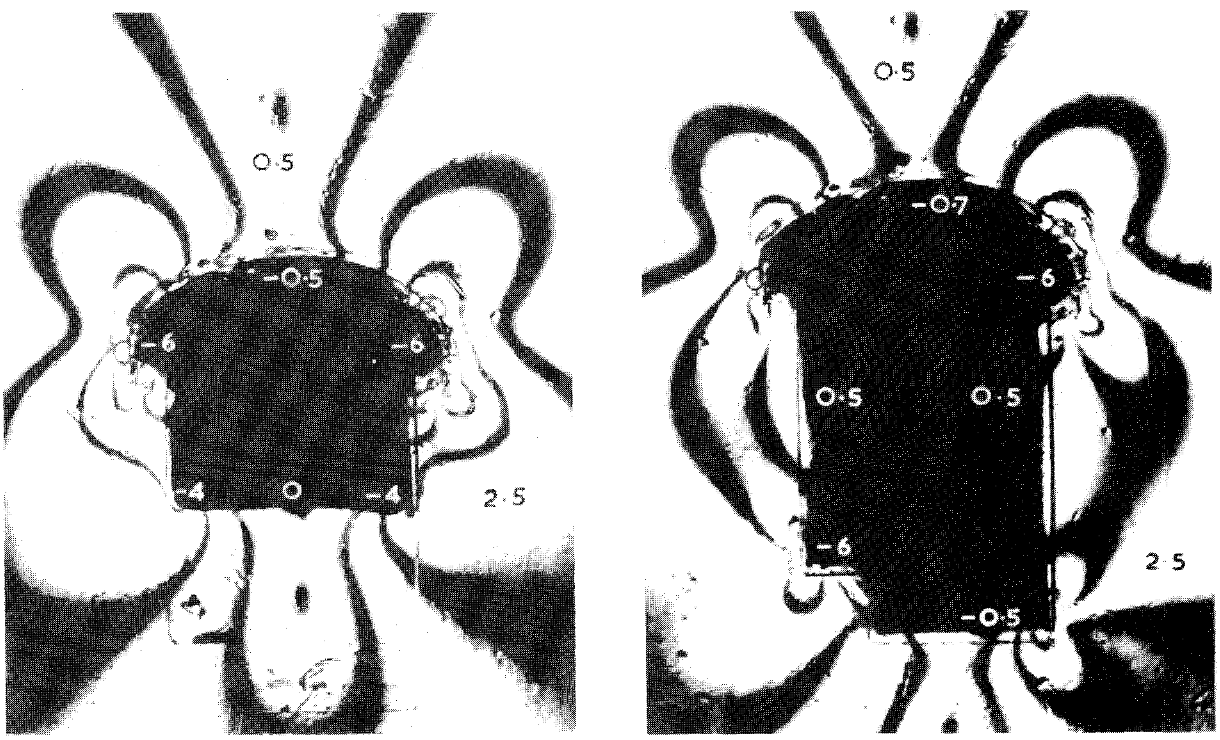


Fig. 7 — Machine hall excavation



(a) $F_n = 270$ p.s.i. $F_s = 135$ p.s.i. (b) $F_n = 250$ p.s.i. $F_s = 125$ p.s.i.

Gravity field $\sigma_V = -1000$ p.s.i. $\sigma_H = 0$ N = 0

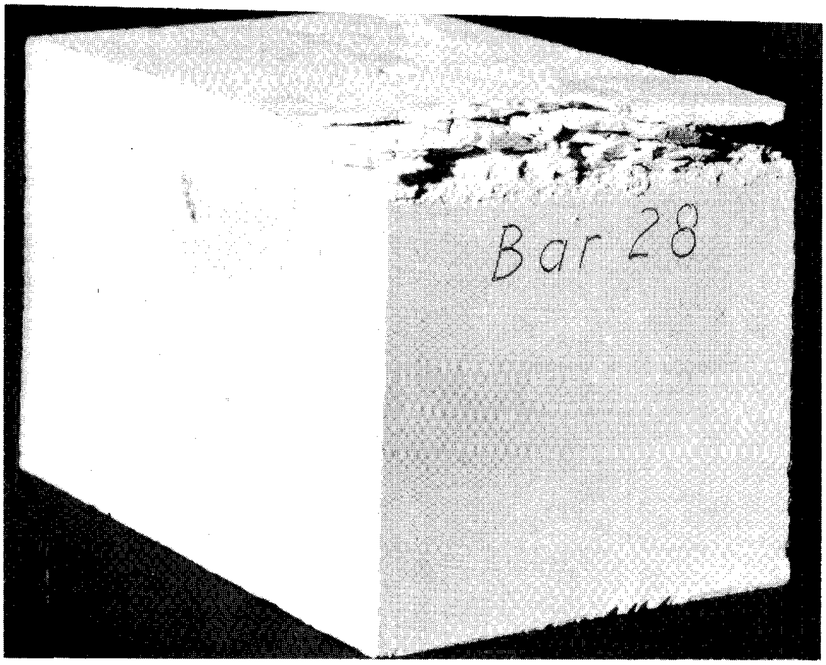


(c) $F_n = 400$ p.s.i. $F_s = 200$ p.s.i. (d) $F_n = 360$ p.s.i $F_s = 180$ p.s.i

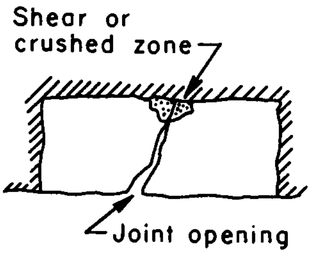
Gravity field $\sigma_V = -1000$ p.s.i $\sigma_H = -400$ p.s.i N = 0.4

Fig. 8 — Excavation sequence gelatin model

(a) Bar under test



(b) Typical failure



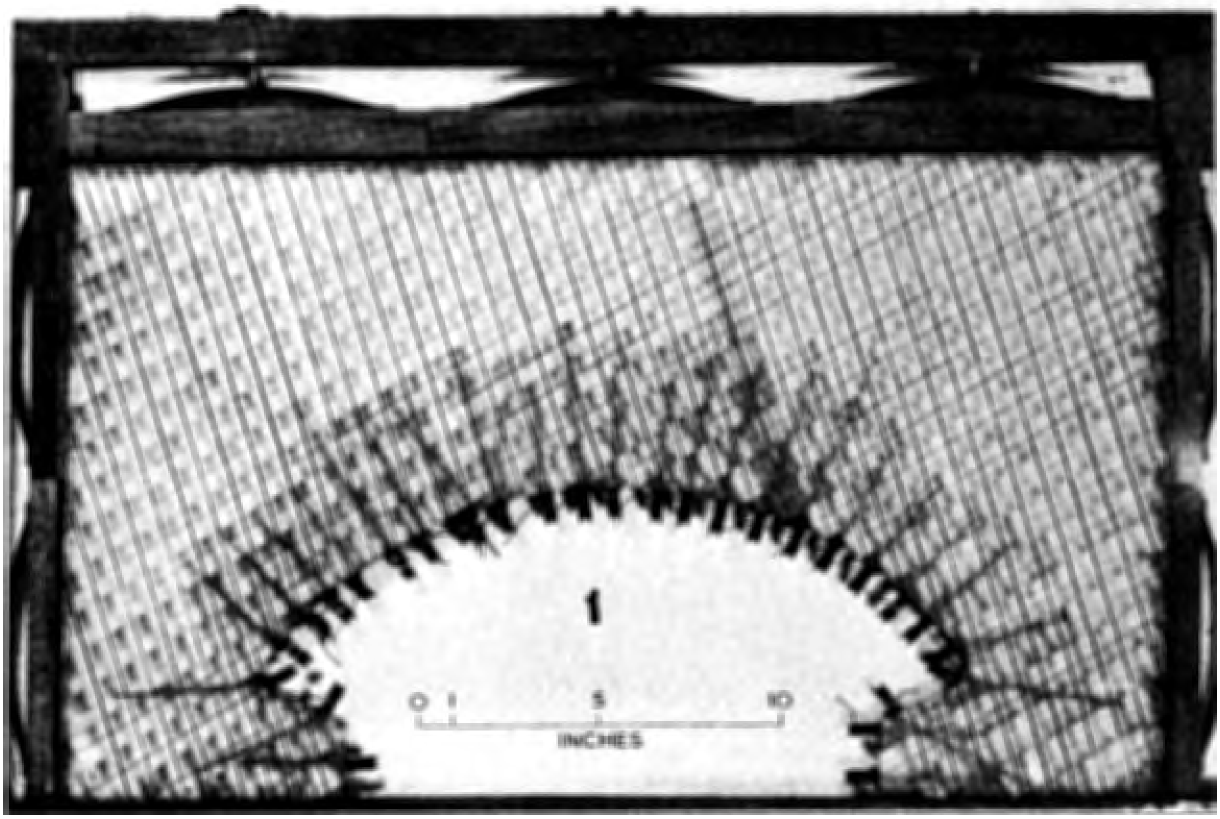
(c) Confined blocks

Fig. 9 — Plaster bar tests of joint failure

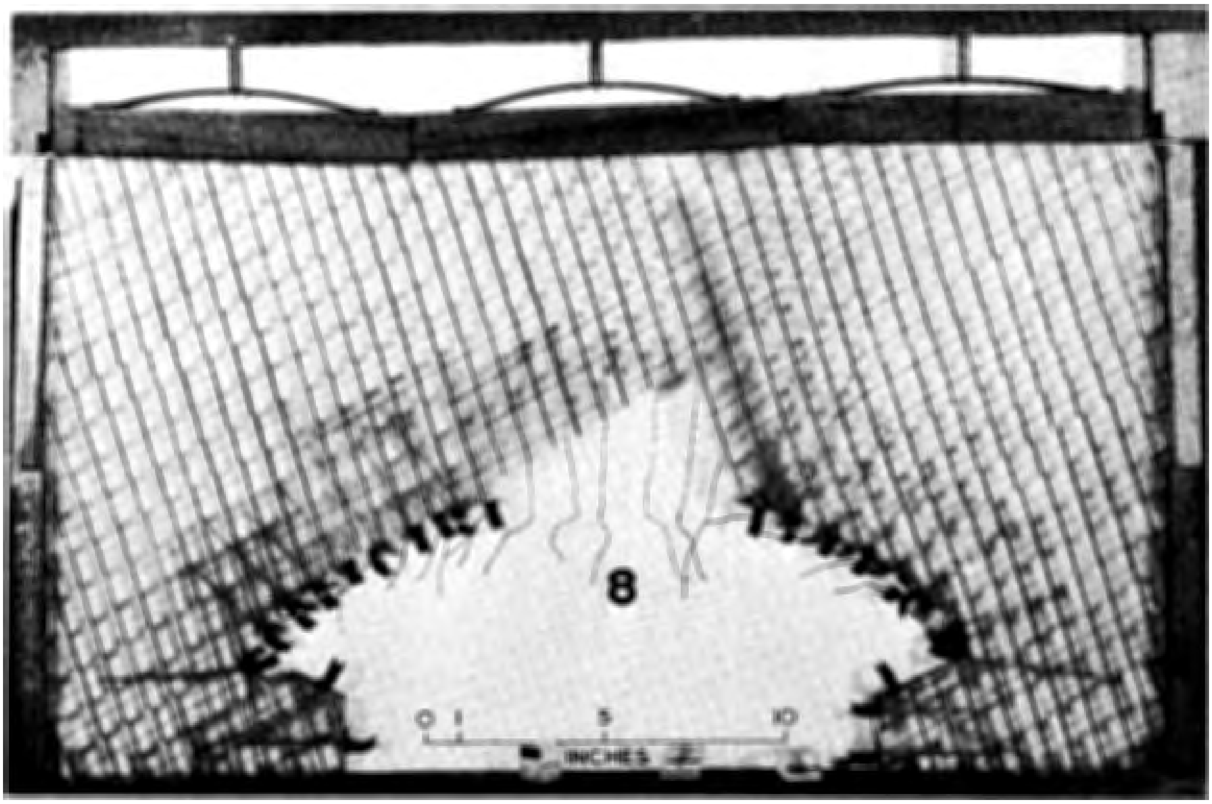
(a) dome roof

(b) side wall

Fig. 10 — Steel rib support in surge tank

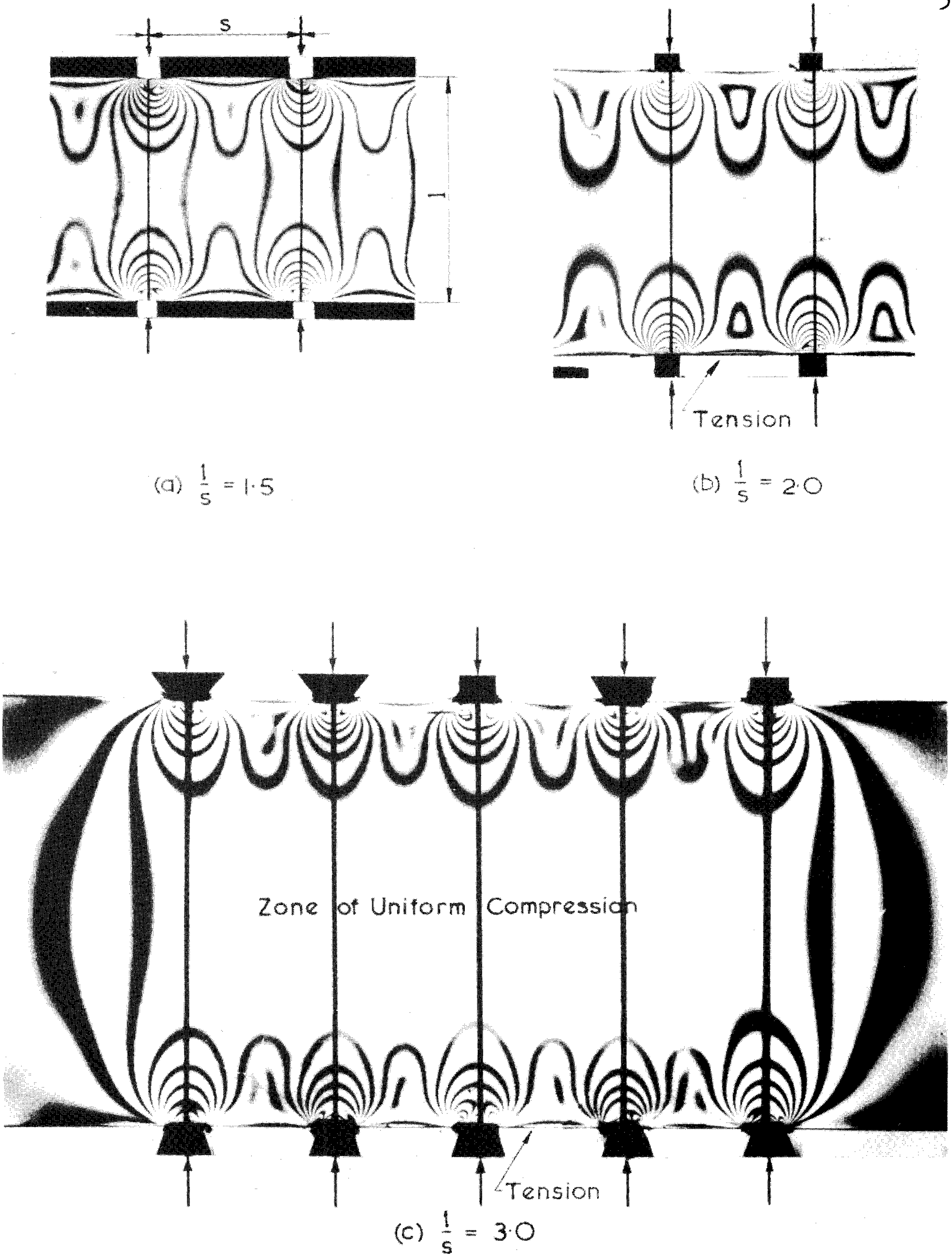


(a)



(b)

Fig. 11 — Block model of machine hall roof



l = Length of Bolt   s = Spacing of Bolts

Fig. 12 — Rock bolt photoelastic stress patterns

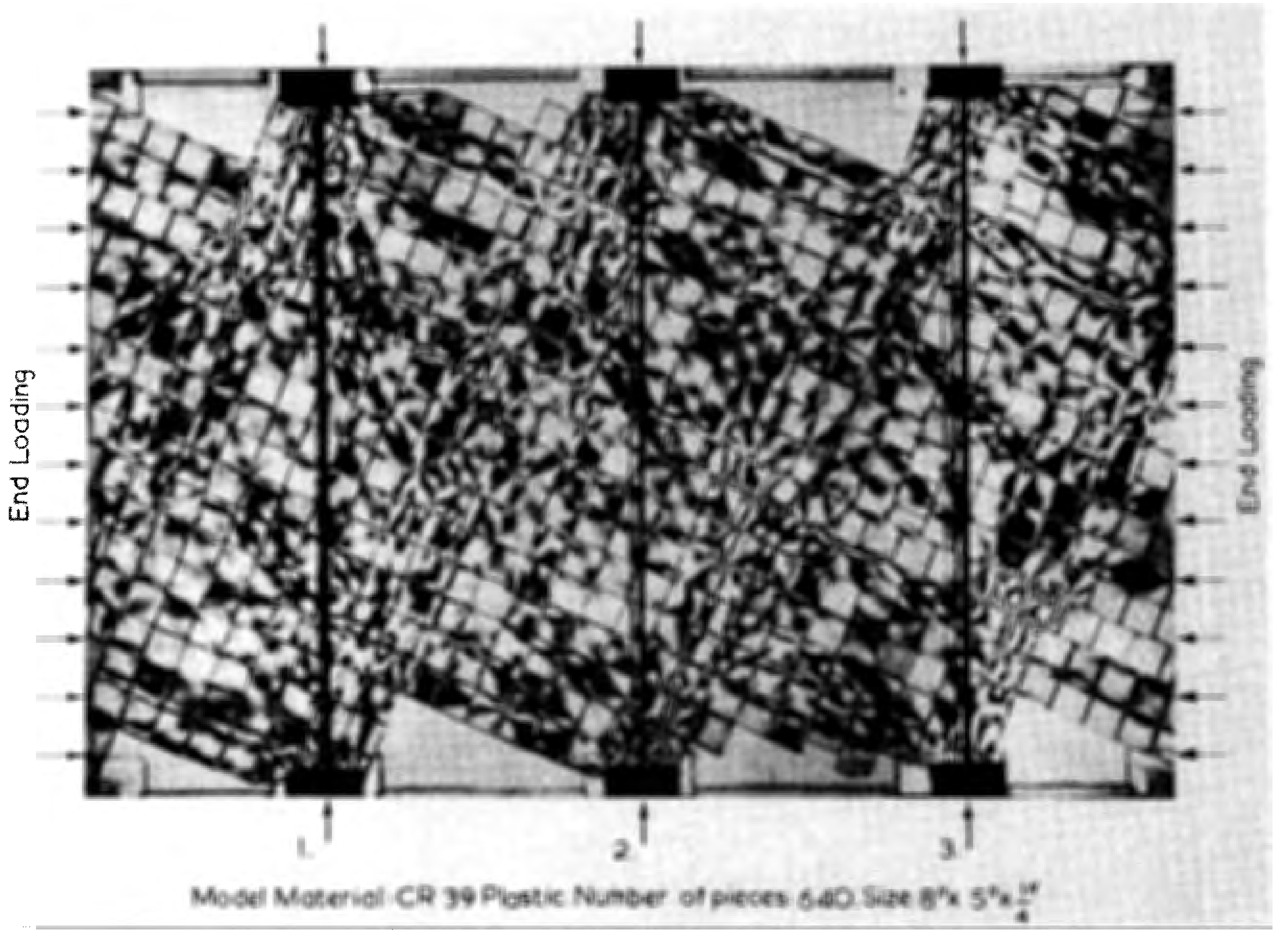
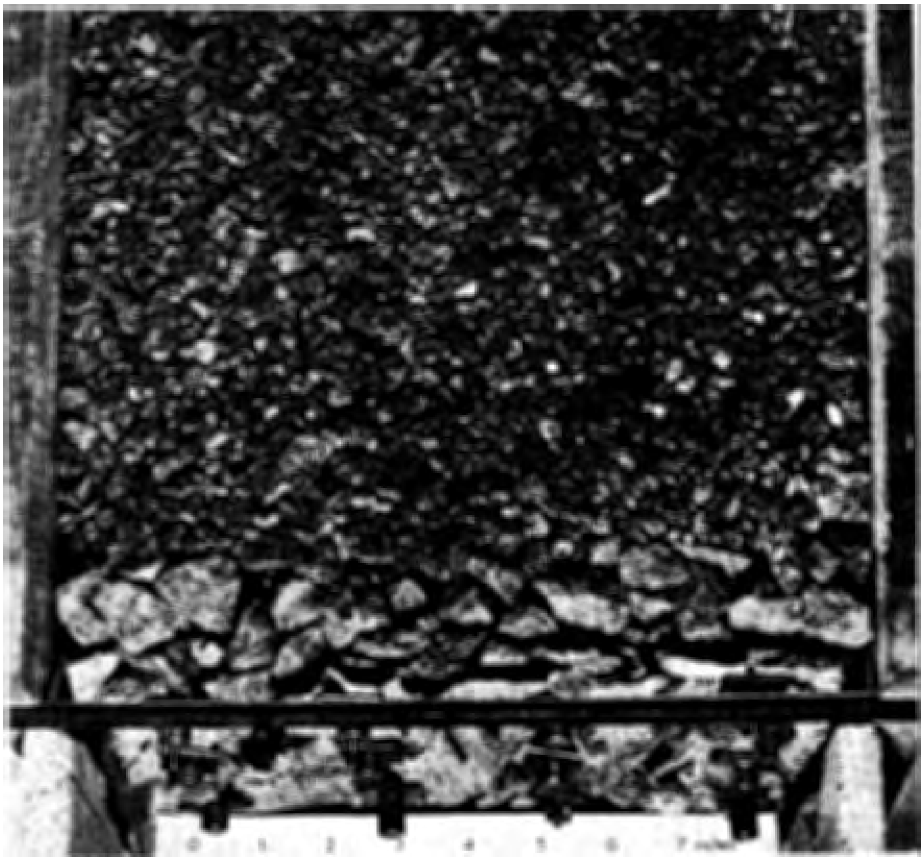
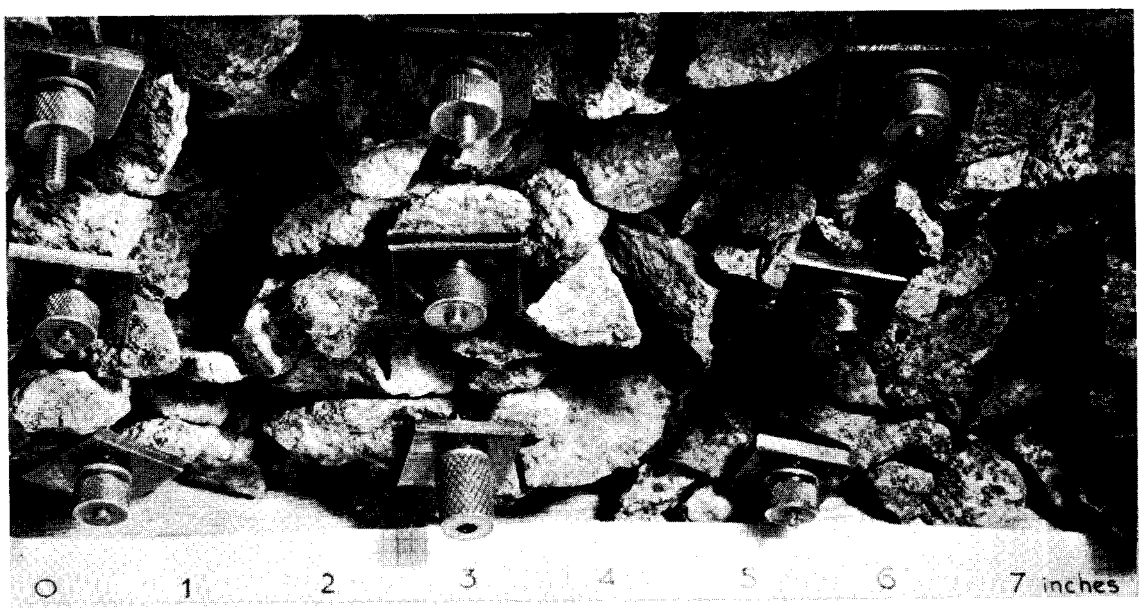


Fig. 13—— Photoelastic rock bolt model of regular joint pattern



(a) Box model



(b) Bolt surface

Fig. 14 — Crushed rock box model

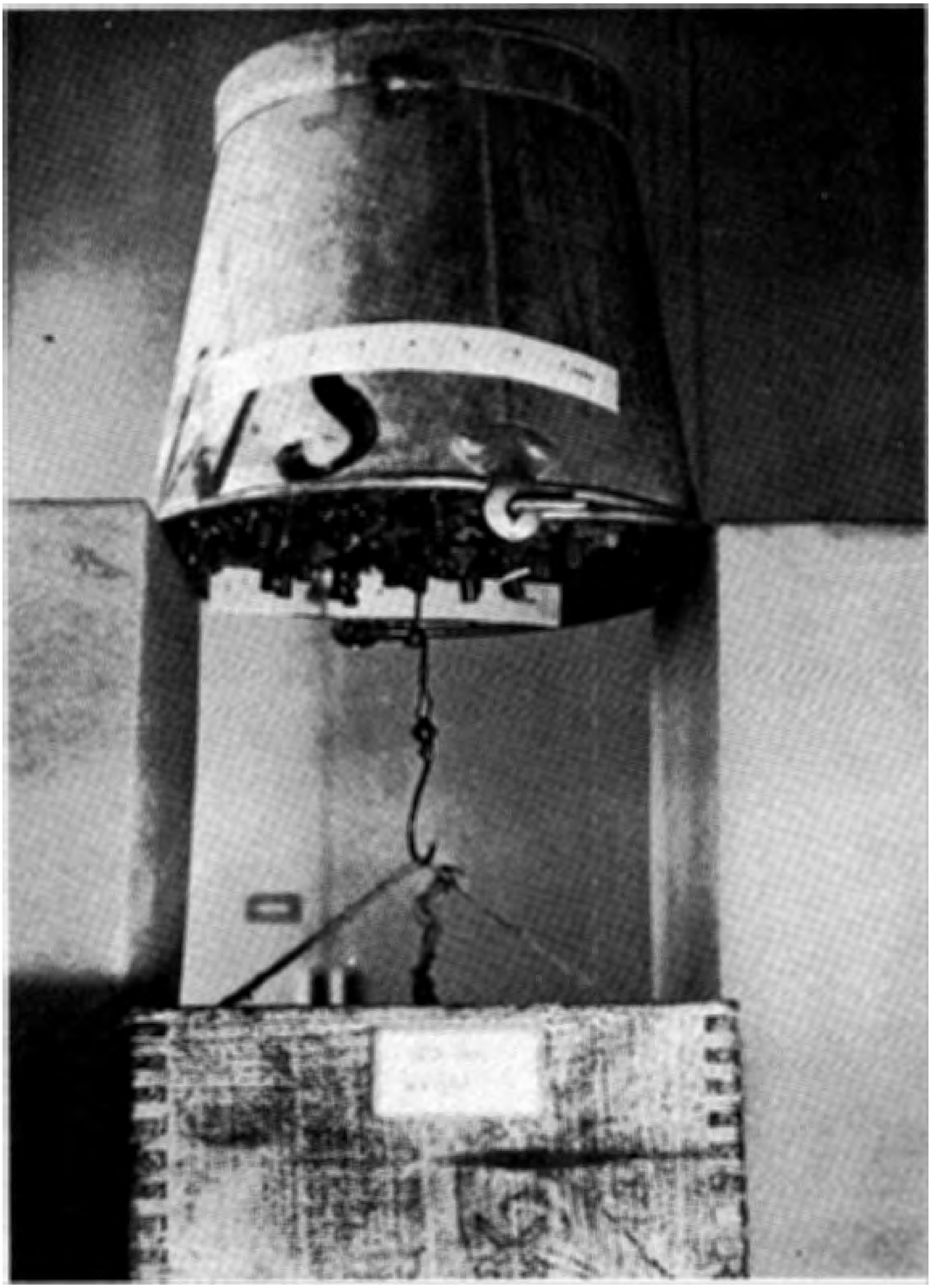


Fig. 15 —— Crushed rock bucket model

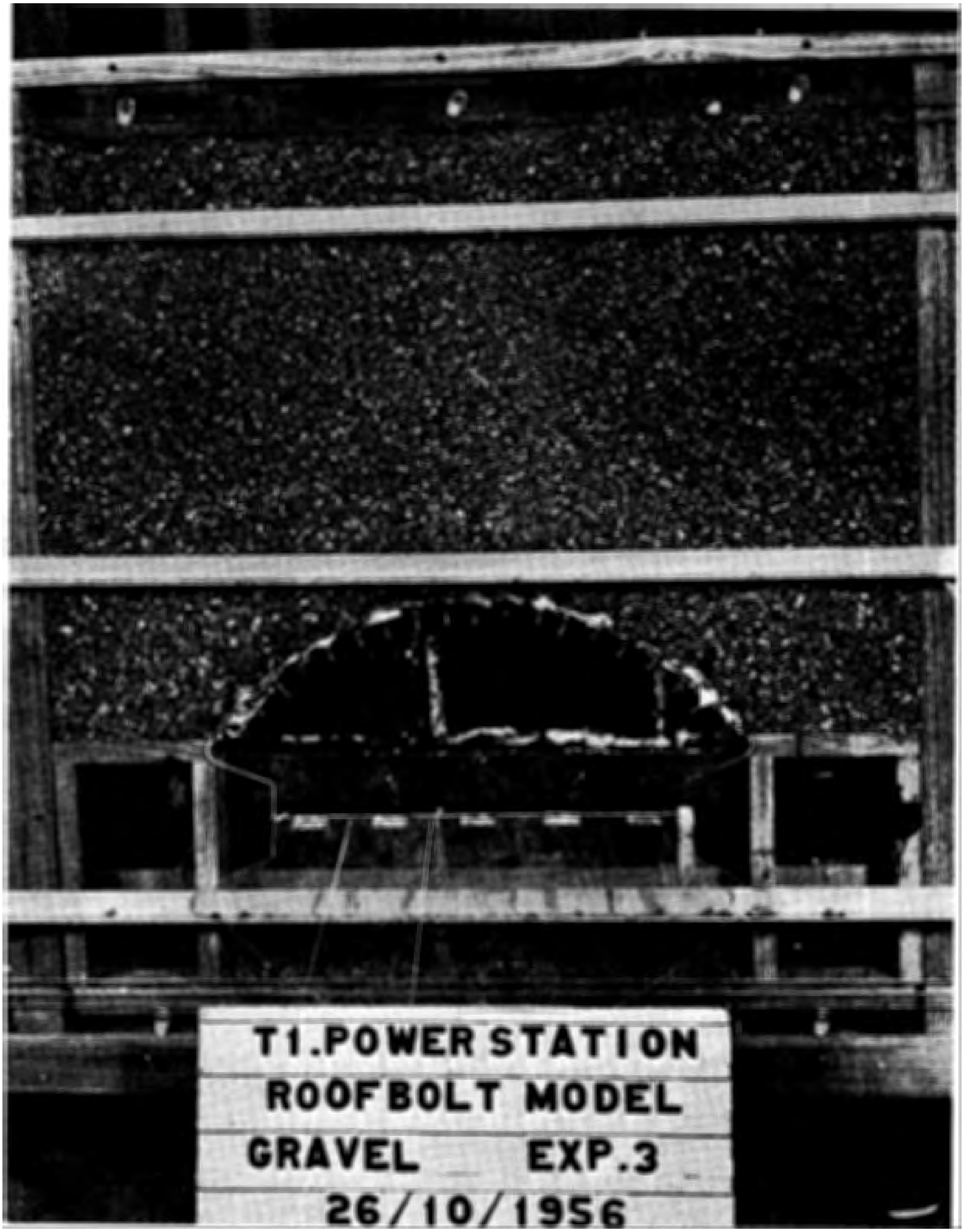
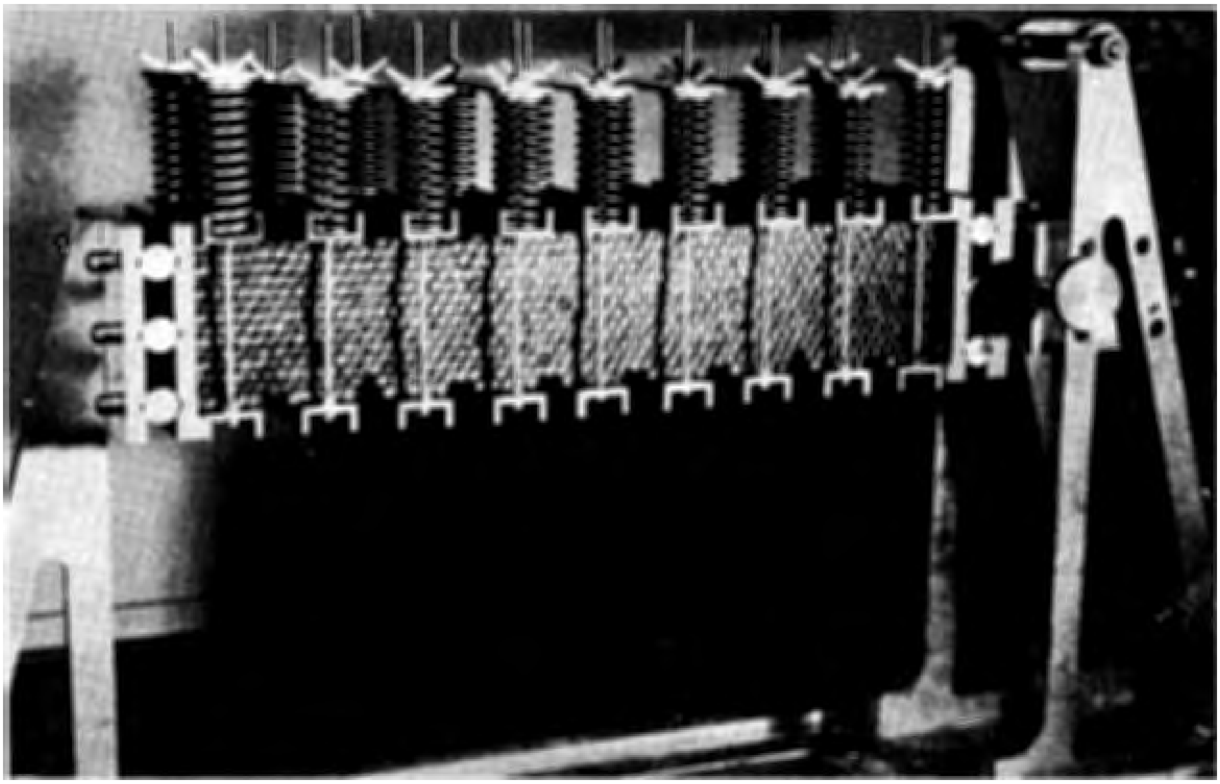
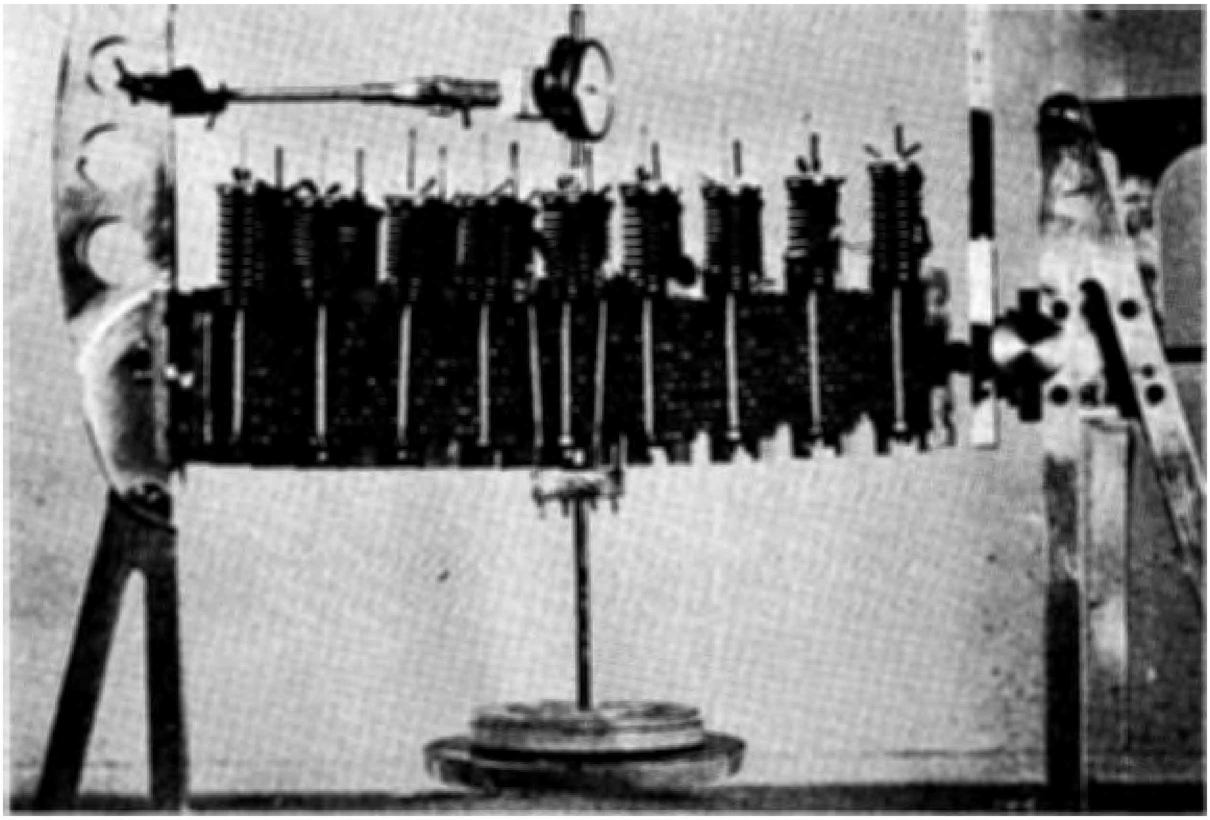


Fig. 16 — Crushed rock model of machine hall roof



(a)



(b)

Fig. 17—Rod model

(a) Crushed rock $1\frac{1}{2}'' - 2\frac{1}{4}''$; 2" x 24 g. wire netting

(b) Wire netting removed at load of 1300 lbs

Fig. 18 —— Rock bolt test

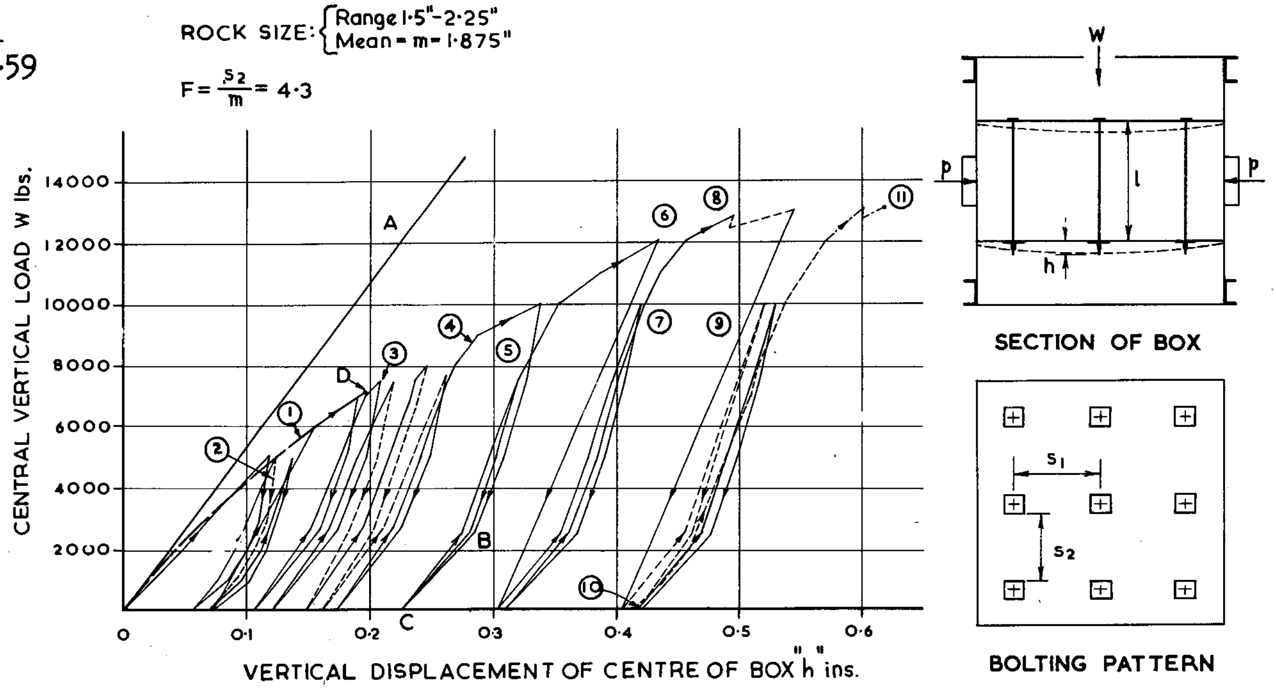


Fig. 19—— Behavior of crushed rock model

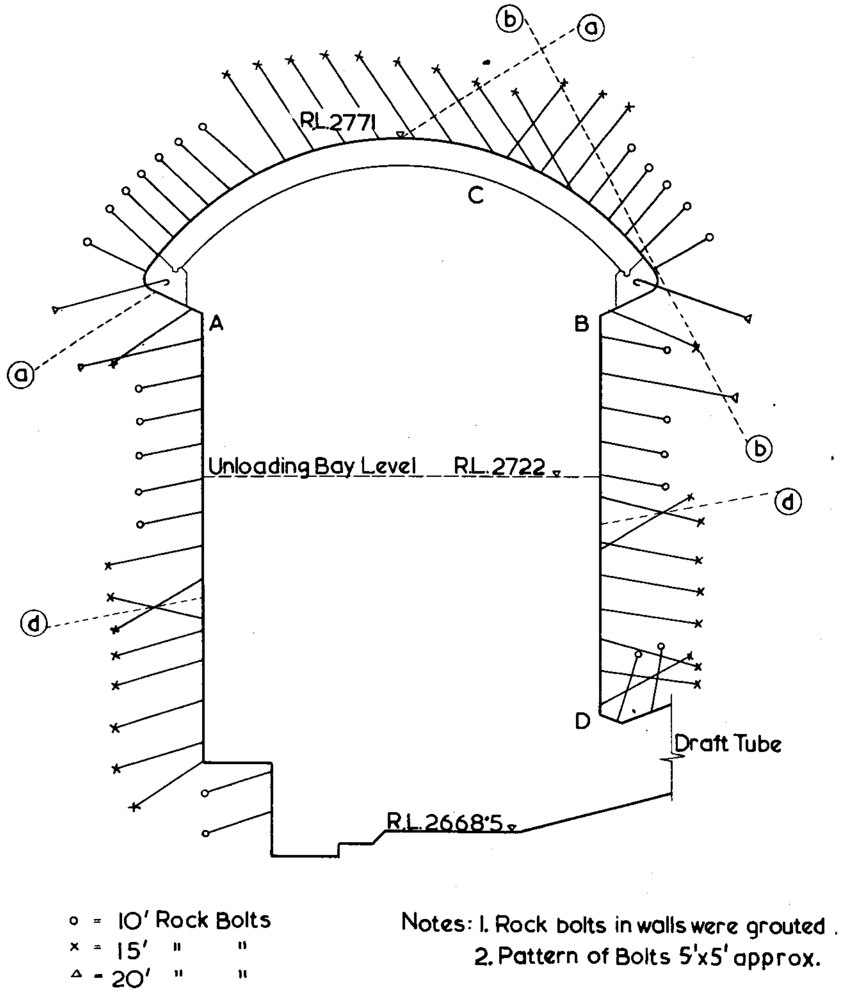
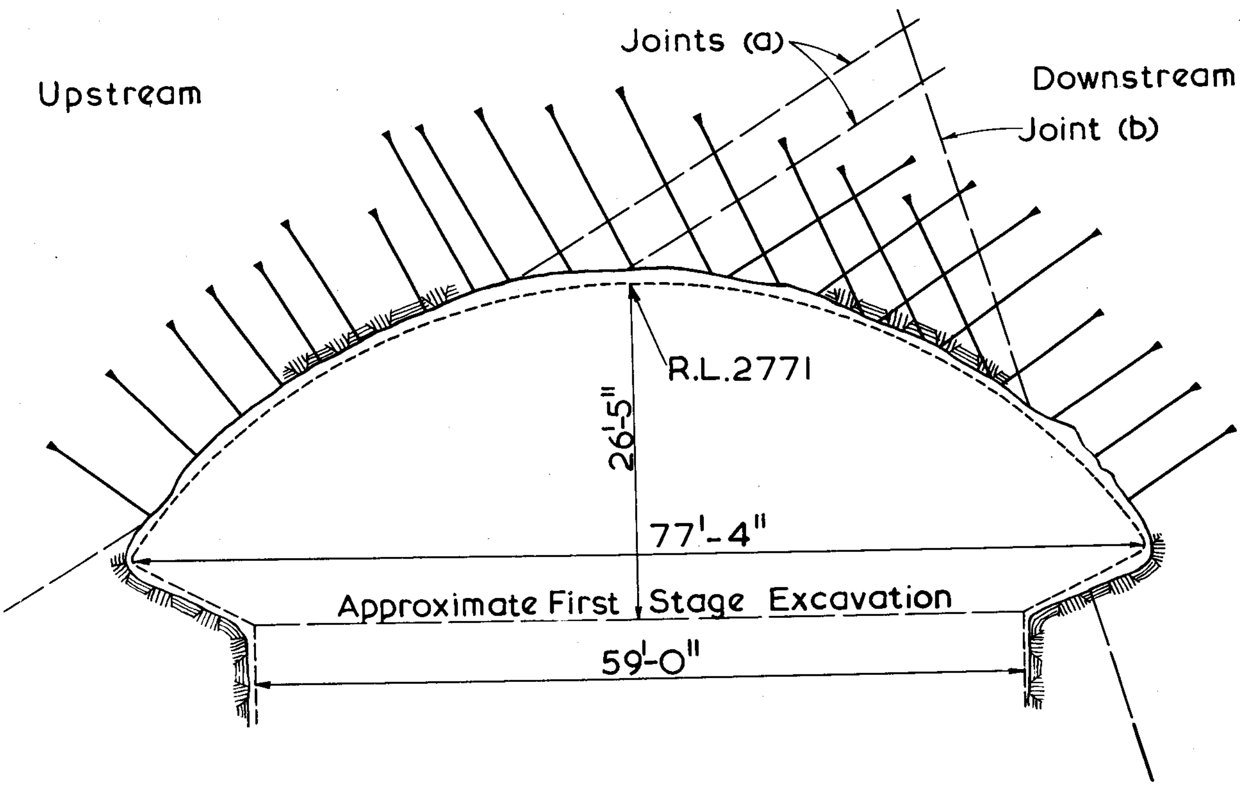


Fig. 20—— Rock bolting in machine hall

(a) Rock bolting and light steel support
September 1956



(b) Rock bolting pattern near rib 16

Fig. 21 — Machine hall support

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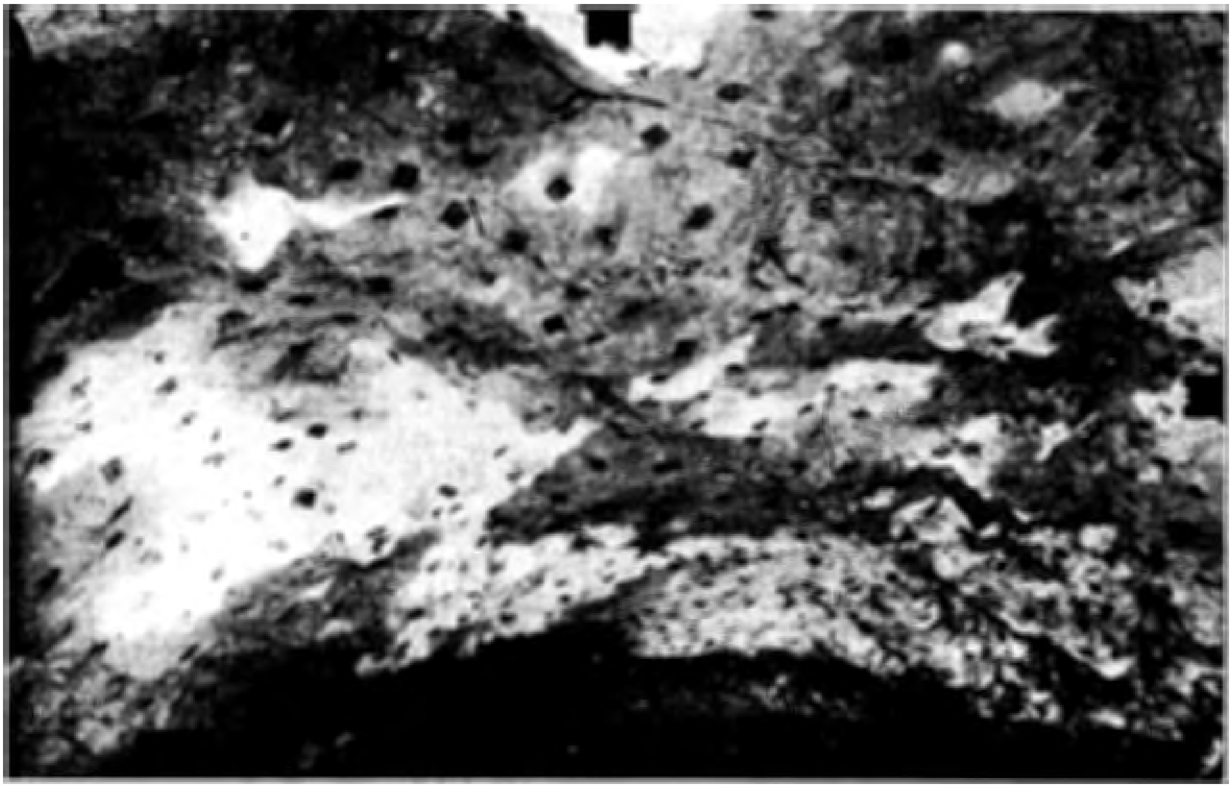
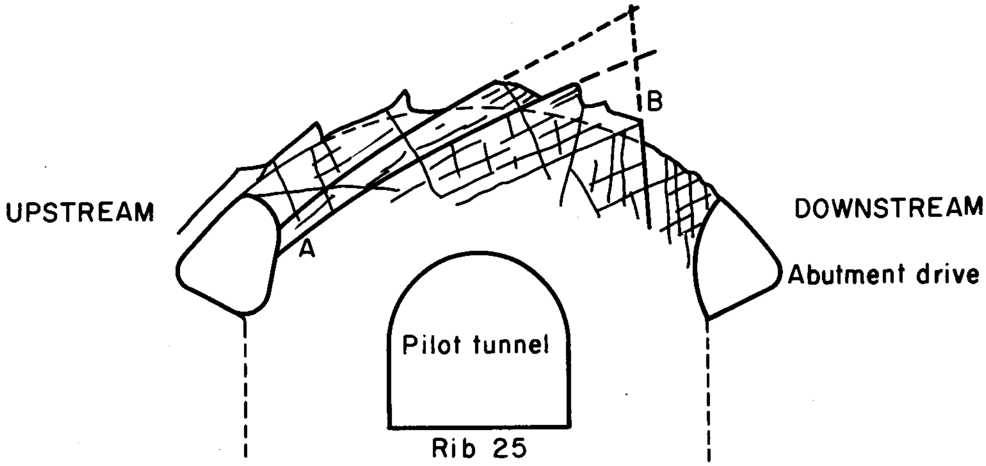


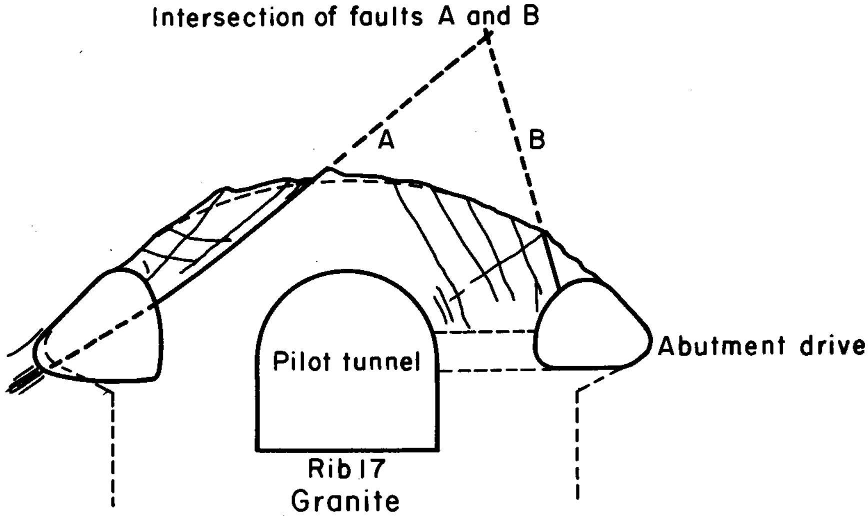
(a) Detail

(b) General view - March 1957

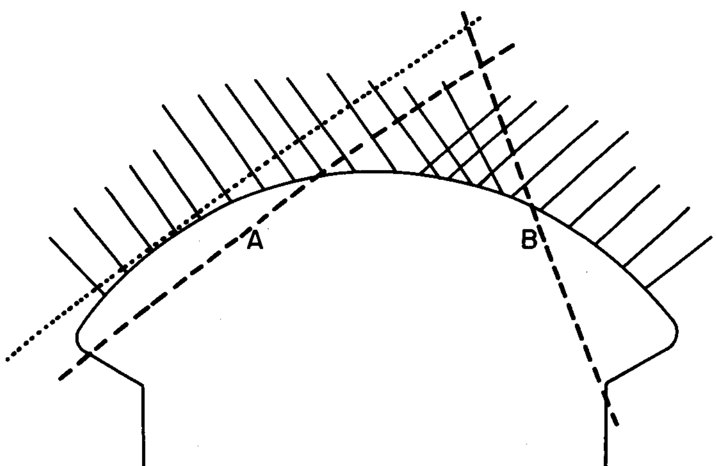
Fig. 22 — Rock bolt support transformer hall



Rib 25
Closely jointed gneiss
Intersection of faults A and B



Rib 17
Granite
Intersection of faults A and B



Rock bolt pattern
In areas affected by faults A and B

Fig. 23—Rock bolt pattern

(a) March 1957

(b) May 1957

Fig. 24 — Machine hall roof

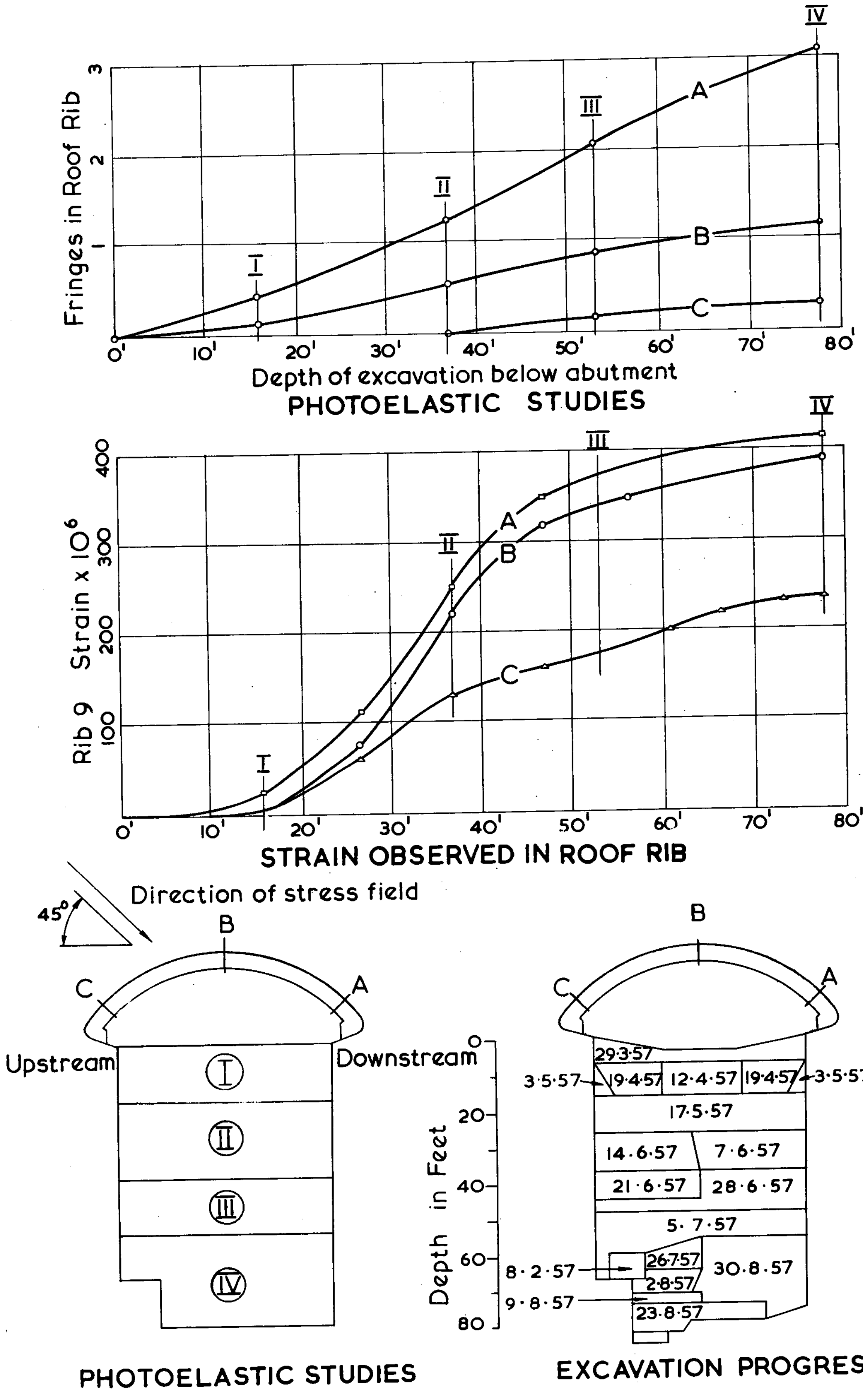


Fig. 25 —— Roof rib behavior

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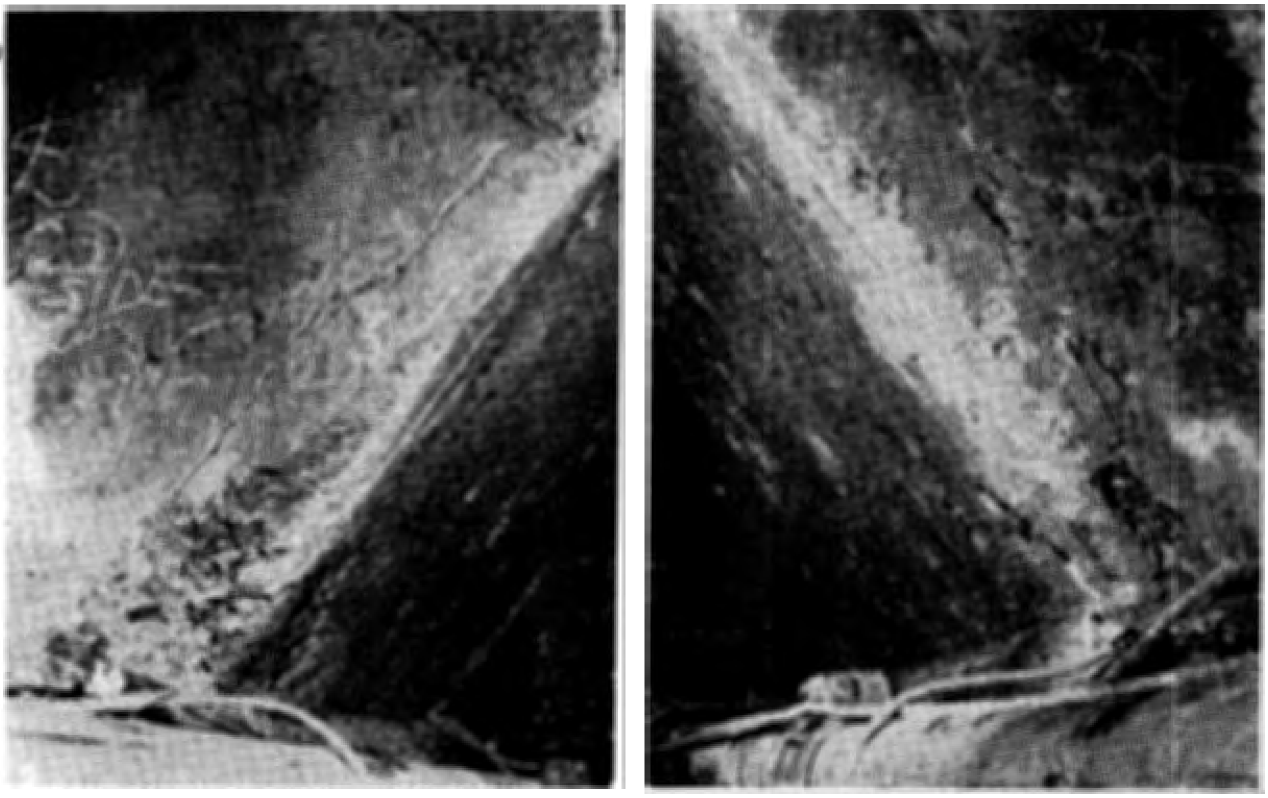
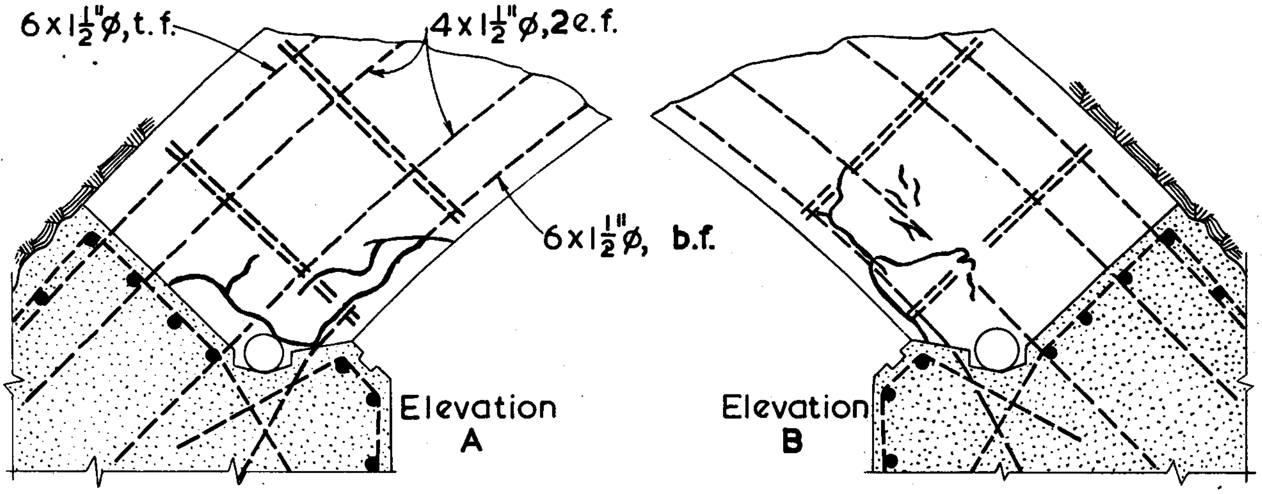
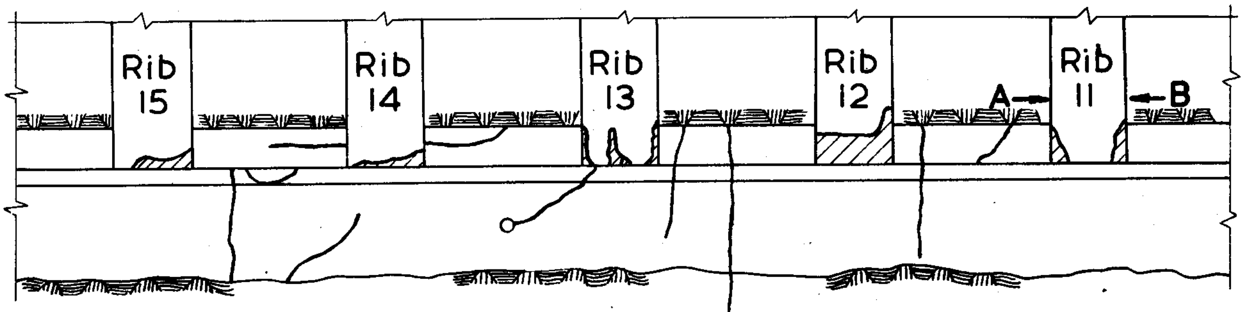




RIB No. 11



DOWNSTREAM ABUTMENT

Fig. 26 — Concrete damage — ribs number 11 - 15

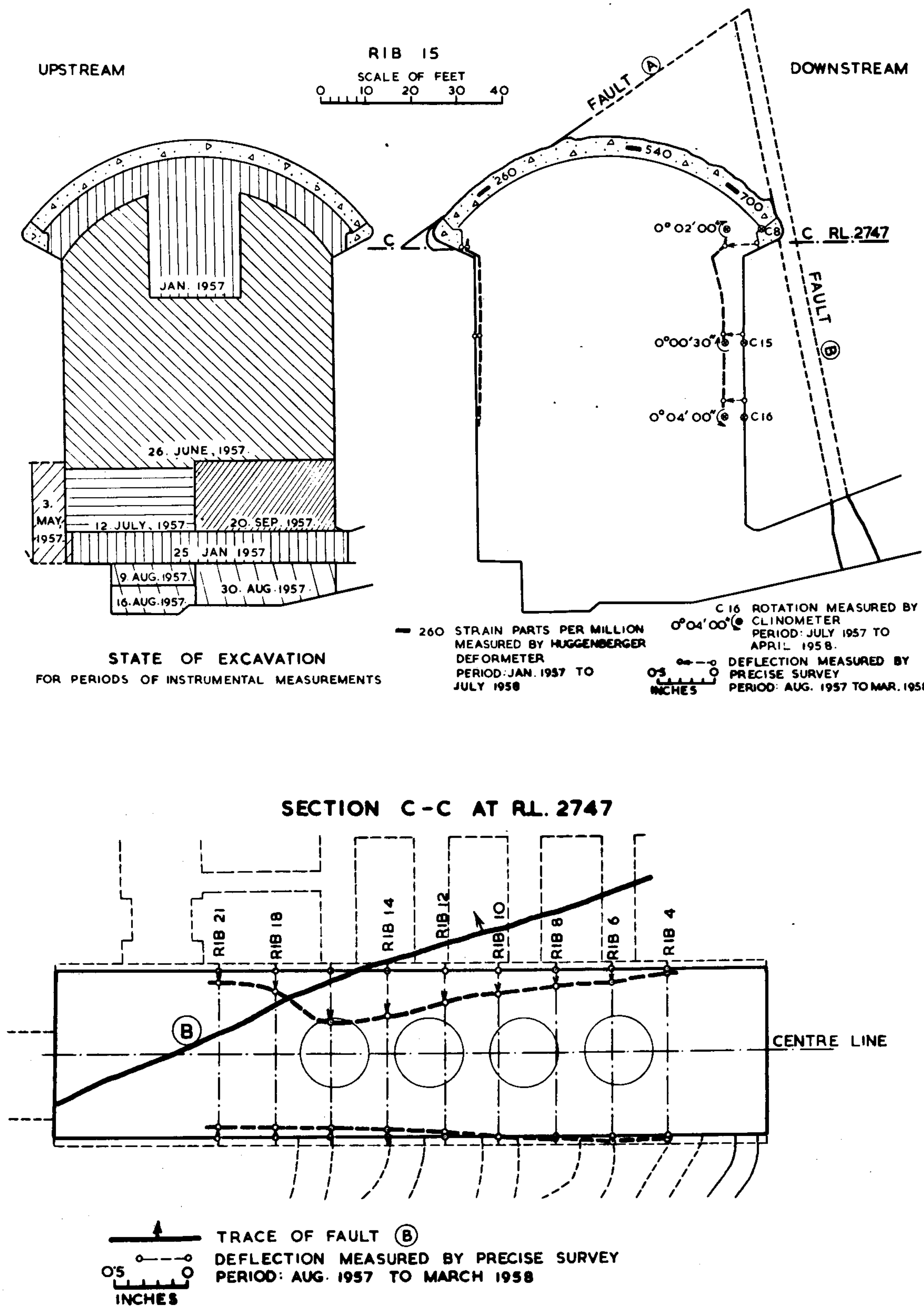


Fig. 27—— Excavation behavior

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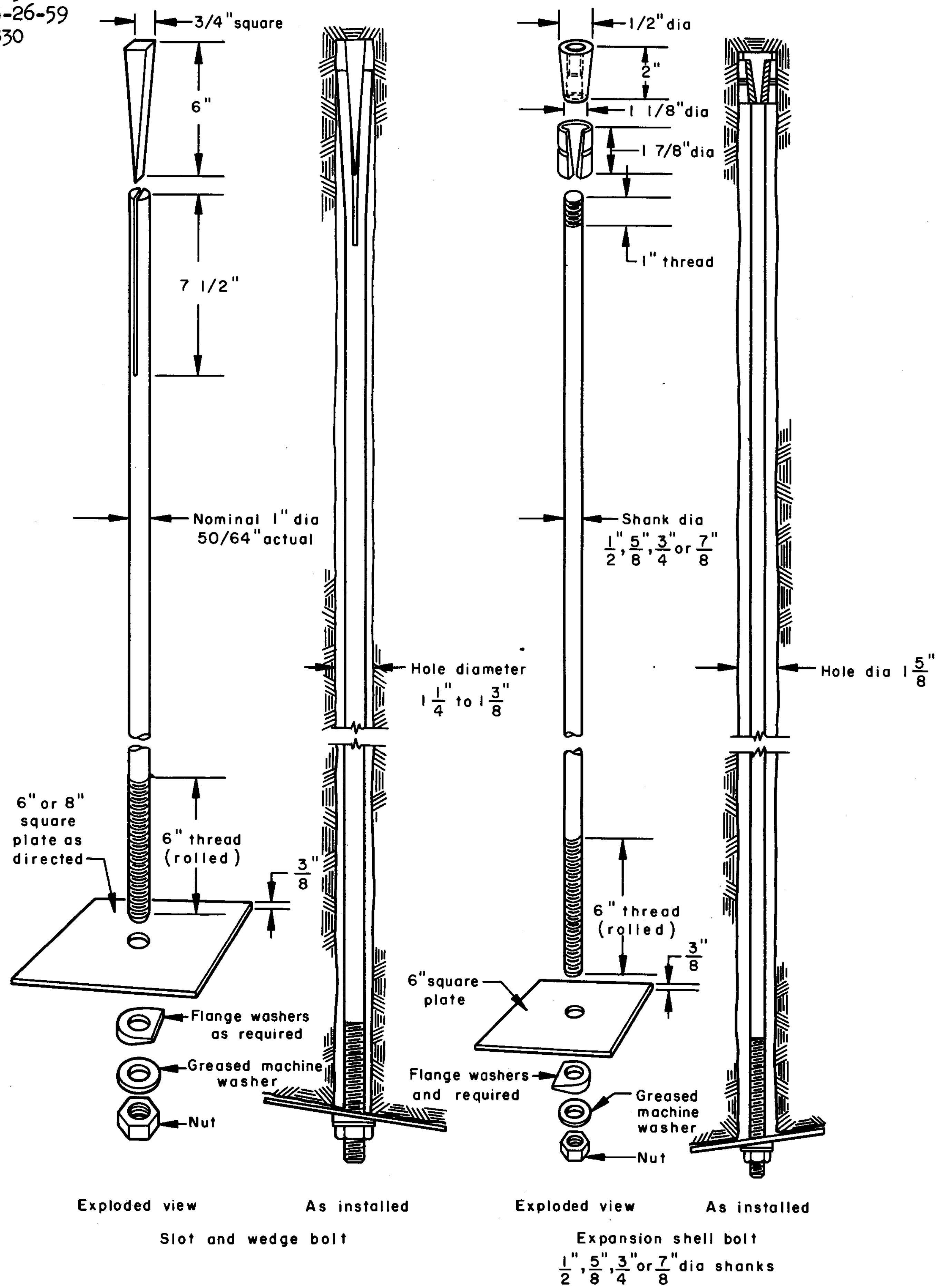

Fig. 28—Types of rock bolts

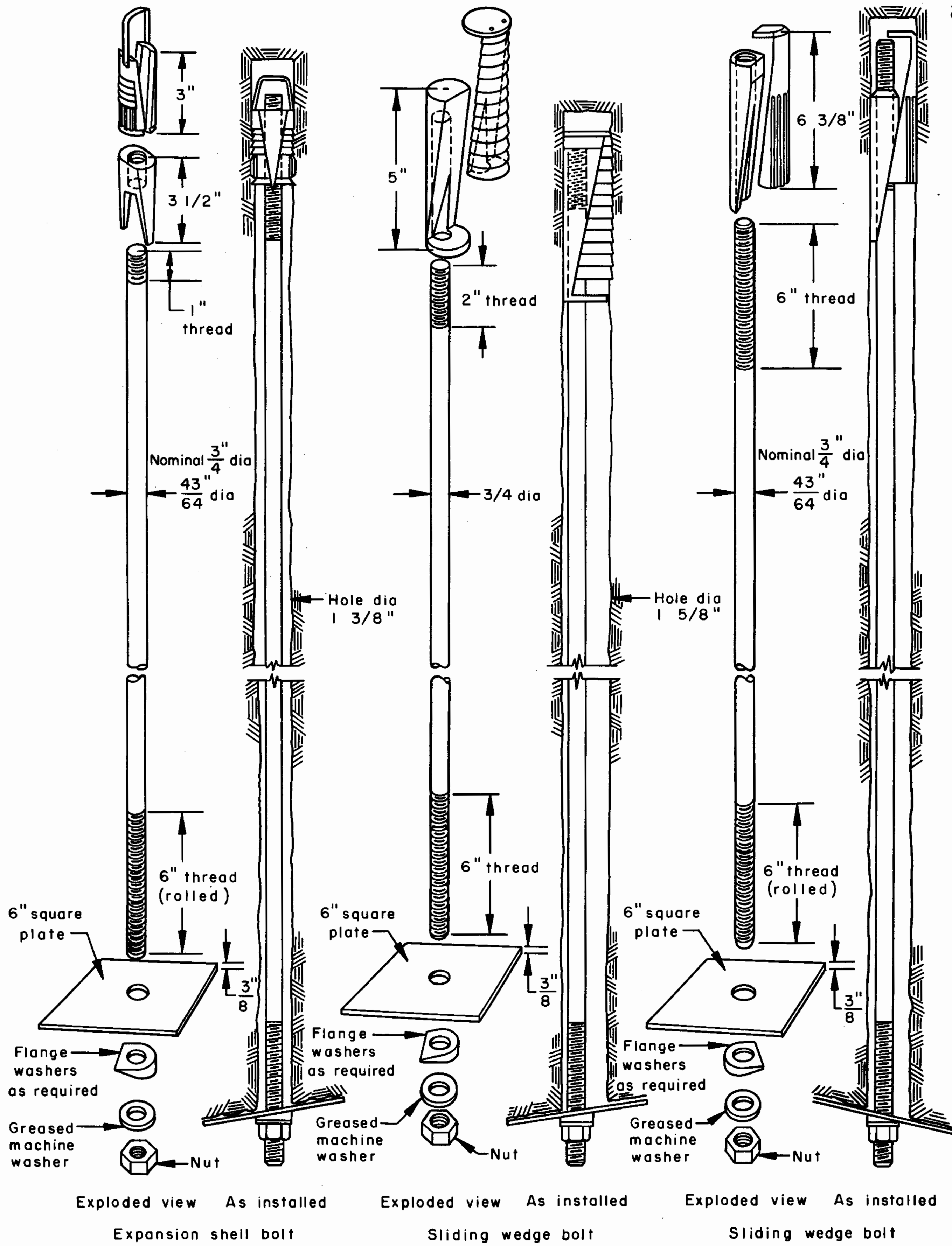

Fig. 29—Types of rock bolts

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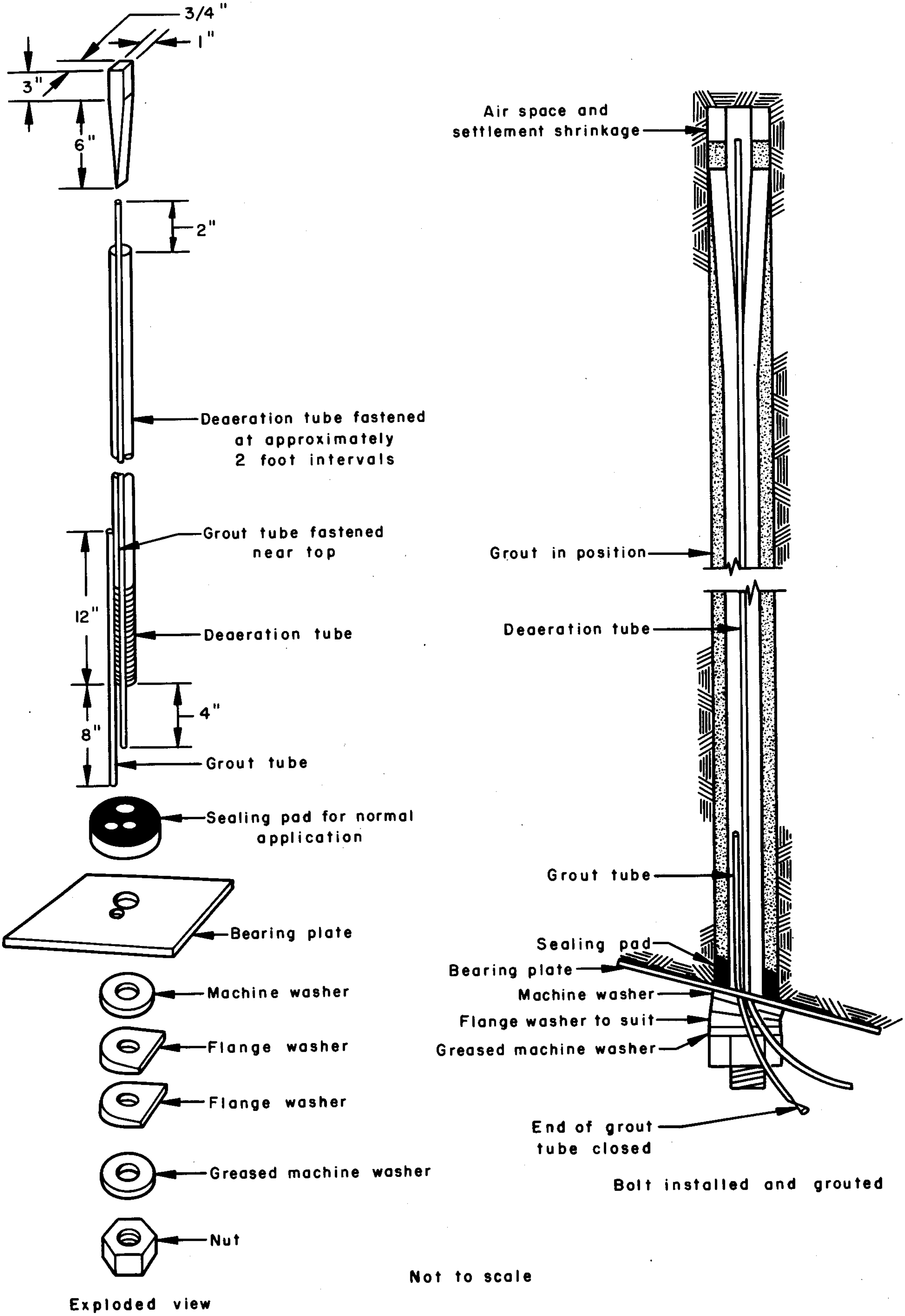


Fig. 30—Grouted rock bolt

APPENDIX I

REFERENCES

(1) "The Upper Tumut Works" by D. E. Campbell, A.N.G. Bray, I. L. Pinkerton, A.C.H. Frost. Journal of the Institution of Engineers, Australia, Jan-Feb. 1956.

(2) "Engineering Geology for the Snowy Mountains Scheme" by D. G. Moye. Journal of the Institution of Engineers, Australia, Oct-Nov. 1955.

(3) "Report on Geology of Upper Tumut Development" by D. G. Moye, Snowy Mountains Hydro-electric Authority, June 1953.

(4) "Rock Behavior and Rock Bolt Support in Large Excavations" by T. A. Lang. Paper presented at Symposium on Underground Power Stations, ASCE Convention, New York, October 1957.

(5) "Rock Mechanics in the Investigation and Construction of T1 Underground Power Station, Snowy Mountains, Australia" by D. G. Moye. Paper presented to Symposium on Engineering Geology, G. S. A. Annual Meeting, St. Louis, November, 1958.

(6) "Stresses in Rock About Cavities" by Karl Terzaghi and F. E. Rickart, Jnr., Geotechnique, Vol. 3, No. 2, 1952.

(7) "Stress Distribution Around a Tunnel" by R. D. Mindlin, A.M. ASCE, Transactions, ASCE, Vol. 105 (1940), p. 117 et seq.

(8) "Elasticity, Fracture, and Flow" by J. C. Jaeger. Methuen and Co. Ltd., London, 1956.

(9) "Theory of Flow and Fracture of Solids" by A. Nadai, Vol. 1. McGraw Hill Book Co.Inc., 1950.

(10) "The Friction and Lubrication of Solids" by F. P. Bowden and D. Tabor. Oxford University Press, 1954.

(11) "Friction and Lubrication" by F. P. Bowden and D. Tabor. Methuen and Co. Ltd., Lond, 1956.

(12) "Microseismic method of determining the stability of underground openings," by L. Obert and W. I. Duvall, 1957. U.S. Bureau of Mines, Bull. 573.

(13) "Measurement of Residual Stress by the Strain Relief Method" by O. J. Olsen, 1957. Second Symposium on Rock mechanics, Quarterly of the Colorado School of Mines, v. 52, No. 3.

(14) "La Mecanique des Roches", by J. Talobre, 1957. Dunod: Paris.

(15) "Rock Defects and Loads on Tunnel Supports" by K. Terzaghi. Section I of rock tunneling with steel supports: by Proctor, R. V., and White, T. L., The Commercial Shearing and Stamping Co., Youngstown, Ohio.

APPENDIX II

ANALYSIS OF ROCK JOINTS AND THE EFFECT OF ROCK BOLTS

by
MILTON A. CHAPPLE, B. E.
(Executive Engineer, Snowy Mountains Authority, Australia)

I. General

The notation used is set out in Section VII.

A tight joint in rock will not affect the stress distribution, and hence will not affect the strength of the rock, provided the joint can transmit the stresses in the rock without relative movement of the rock on either side of it. This requires that

(i) the normal stress σ shall not exceed a certain limiting value which is usually assumed to be zero, but may be slightly positive (i.e., tension on joint) or even a function of the shear stress, and

(ii) the absolute value of the shear stress τ shall not exceed a certain limiting value which may be a function of σ, for example

$$|\tau| < \tau_0 - \mu\sigma \qquad (1)$$

where τ_0 is the cohesion and μ the coefficient of joint friction.

If there is a tendency to exceed either of these limits, there will be in the first case local opening of the joint and in the second case local sliding. In either case, there will be a redistribution of stress to re-establish equilibrium. This need not be regarded as failure of the joint, although it may lead to the development of stresses which will cause premature failure of the rock because of the joint. As the tendency to exceed the limiting stress on the joint increases, the joint will eventually fail either by complete opening or general sliding.

II. Smooth Joint

In the case of a smooth joint, it may be assumed that there will be no opening of the joint if

$$\sigma < 0 \qquad (2)$$

and no sliding if

$$|\tau| < -\mu\sigma \qquad (3)$$

III. Rough Joint

A rough joint with intimate contact is strengthened by the interlocking of the two parts. Let σ_V be the normal stress in a direction parallel to the plane of the joint, and ψ_0 the maximum angle between the rough surface and this plane. (Fig. 1) Then the normal stress on any point of the rough surface is given by

$$\sigma' = \sigma \cos^2\psi + \sigma_V \sin^2\psi + 2\tau \sin\psi \cos\psi \qquad (4)$$

where

$$-\psi_0 < \psi < \psi_0$$

It may be assumed that opening of the joint will occur if $\sigma' > 0$ for all values of ψ in the above range, i.e., if

$$\sigma \cos^2\psi + \sigma_V \sin^2\psi + 2\tau \sin\psi \cos\psi > 0, \quad -\psi_0 < \psi < \psi_0$$

Since, changing the sign of ψ, does not change the first two terms of this expression, the minimum value occurs when τ and ψ have opposite signs. We therefore have

$$\sigma \cos^2\psi + \sigma_V \sin^2\psi - 2|\tau| \sin\psi \cos\psi > 0, \quad 0 < \psi < \psi_0$$

Put $\psi = 0$ Then $\sigma > 0$

Dividing the inequality throughout by $\sin^2\psi$ we have

$$f(\psi) \equiv \sigma \cot^2\psi + \sigma_V - 2|\tau| \cot\psi > 0 \qquad (5)$$

This expression has a minimum at

$$\cot\psi = \frac{|\tau|}{\sigma}$$

If, then, $\frac{|\tau|}{\sigma} > \cot\psi_0$ we require

$$f\left(\frac{|\tau|}{\sigma}\right) \equiv \sigma_V - \frac{\tau^2}{\sigma} > 0$$

and if $\frac{|\tau|}{\sigma} > \cot\psi_0$

$$f(\psi_0) \equiv \sigma \cot^2\psi_0 + \sigma_V - 2|\tau| \cot\psi_0 > 0$$

$$\sigma \cos^2\psi_0 + \sigma_V \sin^2\psi_0 - 2|\tau| \sin\psi_0 \cos\psi_0 > 0$$

Since $\frac{|\tau|}{\sigma} > \cot\psi_0$ implies $\sigma > 0$ the conditions that there be no opening of the joint may be written

$$\left.\begin{array}{ll} \text{(i) if} & \frac{|\tau|}{\sigma} > \cot\psi_0, \quad \sigma_v - \frac{\tau^2}{\sigma} < 0 \\ \text{(ii) if} & \frac{|\tau|}{\sigma} < \cot\psi_0, \quad \sigma < 0, \\ \text{or} & \\ & \sigma\cos^2\psi_0 + \sigma_v\sin^2\psi_0 - 2|\tau|\sin\psi_0\cos\psi_0 < 0 \end{array}\right\} \quad (6)$$

For a rough joint to slide along the plane of the joint, the shear stress must exceed the full shear strength of the material. Adopting the Coulomb-Navier theory of failure we have

$$|\tau| < \tau_0 - \mu\sigma \qquad (7)$$

It is possible, however, for a rough joint to slide in a direction inclined to the joint at an angle ψ so that the proportion of the material to be sheared is reduced to $a \equiv a(\psi)$
The joint will then slide if

$$|\tau'| > a\tau_0 - \mu\sigma'$$

where

$$\sigma' = \sigma\cos^2\psi + \sigma_v\sin^2\psi + 2\tau\sin\psi\cos\psi$$

$$\tau' = \tau(\cos^2\psi - \sin^2\psi) + (\sigma_v - \sigma)\sin\psi\cos\psi$$

$$-\psi_0 < \psi < \psi_0$$

Let a be of the form

$$a = \cos^2\psi + 2a_1 \sin\psi \cos\psi - a_2 \sin^2\psi$$

Then the joint will begin to slide in the direction of positive τ' when, for some value of ψ ,

$$\begin{aligned} \tau(\cos^2\psi - \sin^2\psi) + (\sigma_v - \sigma)\sin\psi\cos\psi &= \\ = \tau_0(\cos^2\psi + 2a_1\sin\psi\cos\psi - a_2\sin^2\psi) &- \\ - \mu(\sigma\cos^2\psi + \sigma_v\sin^2\psi + 2\tau\sin\psi\cos\psi) & \end{aligned} \tag{8}$$

$$\begin{aligned} \cos^2\psi(\tau - \tau_0 + \mu\sigma) + \sin\psi\cos\psi(2\mu\tau - 2a_1\tau_0 + \sigma_v - \sigma) &- \\ - \sin^2\psi(\tau - a_2\tau_0 - \mu\sigma_v) &= 0 \end{aligned}$$

The corresponding value of ψ may be determined by differentiating this equation with respect to ψ and solving the two equations for ψ. If $-\psi_0 < \psi < \psi_0$ the criterion for sliding of the joint may be determined by eliminating ψ between the two equations, when it is found:

$$\begin{vmatrix} \tau - \tau_0 + \mu\sigma & 2\mu\tau - 2a_1\tau_0 + \sigma_v - \sigma & -\tau_0(1-a_2) + \mu(\sigma_v - \sigma) & 0 \\ 0 & -\tau_0(1-a_2) + \mu(\sigma_v + \sigma) & 2\mu\tau - 2a_1\tau_0 + \sigma_v - \sigma & \tau - a_2\tau_0 + \mu\sigma_v \\ 2\mu\tau - 2a_1\tau_0 + \sigma_v - \sigma & -4\tau + 2\tau_0(1+a_2) + 2\mu(\sigma_v - \sigma) & 0 & 0 \\ 0 & 0 & -4\tau + 2\tau_0(1+a_2) + 2\mu(\sigma_v - \sigma) & 2\mu\tau_0 - 2a_1\tau_0 + \sigma_v - \sigma \end{vmatrix} = 0 \tag{9}$$

A similar criterion may be determined for sliding of the joint in the direction of negative τ'.

IV Simple Joint, Bolted

The simple joint studies are confined to two dimensional cases of the kind shown in Figs. 2, 3, and 4. To simplify the presentation resultant forces on a block have been used, viz. P represents the resultant of the forces on the joint parallel to the surface, B represents the force of either one rock bolt or the resultant of several rock bolts, and W is the weight of a discrete block. α is the angle which the joint makes with the normal to the surface or the angle which the normal to a joint makes with the direction of P, and μ or tan ϕ is the coefficient of friction for the joint surface. It should be noted that μ is not the so-called coefficient of internal friction of the rock. It is assumed in these studies that the rock bolt cannot take any shear and exerts only compression in its axial direction.

Consider a block of depth 2c (Fig 2) with a smooth plane joint whose normal is inclined to the axis of the beam at an angle α. Suppose that the loads applied to the beam are such as to produce at the joint an axial thrust P, a transverse shear V and a moment M, and that in addition, due to bolting, a compressive force B acts across the center of the joint at an angle $\frac{\pi}{2}-\theta$ to the axis of the beam. (Fig. 2).

Taking moments about that end of the joint where the moment tends to produce compressive stresses, we find that the joint will not open if

$$|M| - Pc + Vc\tan\alpha - B(c\sin\theta + c\cos\theta\tan\alpha) < 0$$

$$B\,\frac{\sin(\alpha+\theta)}{\cos\alpha} > \left|\frac{M}{c}\right| - P + V\tan\alpha \qquad (10)$$

In particular, if $\theta = 0$

$$B\tan\alpha > \left|\frac{M}{c}\right| - P + V\tan\alpha \qquad (11)$$

and if $\theta = \frac{\pi}{2} - \alpha$,

$$B\sec\alpha > \left|\frac{M}{c}\right| - P + V\tan\alpha \qquad (12)$$

The normal tension on the joint is

$$S = -P\cos\alpha + V\sin\alpha - B\sin(\alpha+\theta)$$

and the shear force is

$$T = P\sin\alpha + V\cos\alpha - B\cos(\alpha+\theta)$$

There will be no sliding, therefore, if

$$|P\sin\alpha + V\cos\alpha - B\cos(\alpha+\theta)| < -\mu(-P\cos\alpha + V\sin\alpha - B\sin(\alpha+\theta))$$

$$\left|P\tan\alpha + V - B\,\frac{\cos(\alpha+\theta)}{\cos\alpha}\right| < \mu\left(P - V\tan\alpha + B\,\frac{\sin(\alpha+\theta)}{\cos\alpha}\right) \qquad (13)$$

In particular, if $\theta = 0$

$$|P \tan\alpha + V - B| < \mu(P - V\tan\alpha + B\tan\alpha) \tag{14}$$

and if $\theta = \frac{\pi}{2} - \alpha$

$$|P\tan\alpha + V| < \mu(P - V\tan\alpha + B\sec\alpha) \tag{15}$$

The condition for no opening of the joint, i.e., no rotation about 0 may be written

$$P - V\tan\alpha + B\,\frac{\sin(\alpha+\theta)}{\cos\alpha} > \left|\frac{M}{c}\right| \tag{16}$$

and for no sliding

$$P - V\tan\alpha + B\,\frac{\sin(\alpha+\theta)}{\cos\alpha} > \frac{1}{\mu}\left|P\tan\alpha + V - B\,\frac{\cos(\alpha+\theta)}{\cos\alpha}\right| > 0 \tag{17}$$

If the moment M is small then the criterion for sliding prevails. The criterion for rotation about 0 applies for a completely open joint w ith the reactions at 0 bearing on a "point" area. Actually before limit 16 is reached failure will occur by shearing or crushing at 0.

V Simple Joint - Applications

1. Simple joint with axial thrust P, bolt normal to joint. (Fig 3 (a))

We have

$$V = 0,\ M = 0,\ \theta = \frac{\pi}{2} - \alpha$$

For stability, therefore,

$$|P\tan\alpha| < \mu(P + B\sec\alpha)$$

and if P is positive

$$P \tan\alpha < \mu(P + B\sec\alpha)$$
$$\frac{B}{P} > \frac{\cos\alpha}{\mu}(\tan\alpha - \mu)$$
$$> \sin\alpha(\cot\phi - \cot\alpha) \qquad (18)$$

where

$$\mu = \tan\phi$$

If $\alpha < \phi$ the right hand side is negative and the bolt is unnecessary for stability

2. Single joint with axial thrust P, bolt normal to free surface Fig. 3 (b)

We have

$$V = 0, \ M = 0, \ \theta = 0$$

For stability, therefore,

$$|P\tan\alpha - B| < \mu(P + B\tan\alpha)$$
$$\mu(P + B\tan\alpha) > P\tan\alpha - B > -\mu(P + B\tan\alpha)$$
$$-P(\tan\alpha - \mu) + B(1 + \mu\tan\alpha) > 0 > -P(\tan\alpha + \mu) + B(1 - \mu\tan\alpha)$$

and if $P, \ 1 - \mu\tan\alpha$ are positive,

$$-\tan(\alpha - \phi) + \frac{B}{P} > 0 > -\tan(\alpha + \phi) + \frac{B}{P}$$
$$\tan(\alpha - \phi) < \frac{B}{P} < \tan(\alpha + \phi) \qquad (19)$$

If $1 - \mu\tan\alpha$ is negative, i.e., if $\alpha + \phi > \frac{\pi}{2}$ the right hand inequality does not apply.

There is an upper and a lower limit for the ratio of B to P. If the tension in the bolt is too high, the bolt can cause the blocks on either side of the joint to slide relative to one another. In the case of a surface block confined on all sides except the free surface this is not a danger because excess tension in the bolt will only cause an increase in the value of P, although some small movement may occur along the joint.

3. Symmetrical double joint in horizontal roof with axial thrust, P, transverse force due to weight W of block between joints, bolts normal to axis of beam. (Fig. 3 (c)).

We have $V = \frac{W}{2}, \; M = 0, \; \theta = 0$

For stability, therefore,

$$|P \tan\alpha + \tfrac{W}{2} - B| < \mu(P - \tfrac{W}{2}\tan\alpha + B\tan\alpha)$$

and as above, if P and $1 - \mu\tan\alpha$ are positive,

$$\tan(\alpha - \phi) < \frac{B + \frac{1}{2}W}{P} < \tan(\alpha + \phi) \qquad (20)$$

Again the right hand side does not apply if $(\alpha + \phi) > \frac{\pi}{2}$. It is rarely that the two joints forming the block W will be at equal angles α to the surface. A criterion for stability can be worked for α different in the two joints. However, for simplicity, they have been kept the same. Under certain conditions the block is quite stable and bolting is **unnecessary.**

4. Symmetrical double joint in vertical surface with axial thrust P across upper joint and $P + W$ across lower joint, W the weight of the block between joints, bolts normal to axis of column.

We have $V = 0, \; M = 0, \; \theta = 0$

For stability of the upper joint, therefore,

$$|P \tan\alpha - B| < \mu(P + B\tan\alpha)$$

$$\tan(\alpha - \theta) < \frac{B}{P} < \tan(\alpha + \phi) \qquad (21)$$

and for stability of the lower joint

$$|(P+W)\tan\alpha - B| < \mu\,(P+W+B\tan\alpha)$$

$$\tan(\alpha-\phi) < \frac{B}{P+W} < \tan(\alpha+\phi) \qquad (22)$$

so that, for stability of both joints

$$\tan(\alpha-\phi) < \frac{B}{P+W}\,, \quad \frac{B}{P} < \tan(\alpha+\phi) \qquad (23)$$

Suppose

$$\frac{B}{P+W} < \tan(\alpha-\phi) < \frac{B}{P}$$

Then the lower joint will tend to slide, and there will be a redistribution of stress to re-establish equilibrium. Suppose that the redistribution is such that the axial thrust is invariant and that a shear V is introduced. Then

$$(P+W)\tan\alpha + V - B = \mu\,(P+W-V\tan\alpha + B\tan\alpha)$$

$$V = B - (P+W)\,\frac{\tan\alpha-\mu}{1+\mu\tan\alpha}$$

$$= B - (P+W)\tan(\alpha-\phi)$$

On the upper joint, therefore, we have

$$V = -B + (P+W)\tan(\alpha-\phi)$$

and for stability

$$P\tan\alpha + (P+W)\tan(\alpha-\phi) - 2B < \mu\,(P - (P+W)\tan(\alpha-\phi)\tan\alpha + {}$$
$$+ 2B\tan\alpha)$$

$$2B(1+\mu\tan\alpha) > P(\tan\alpha-\mu) + (P+W)\tan(\alpha-\phi)(1+\mu\tan\alpha)$$

$$2B > P\tan(\alpha-\phi) + (P+W)\tan(\alpha-\phi)$$

$$\frac{B}{P+\frac{W}{2}} > \tan(\alpha-\phi)$$

Similarly, if $\frac{B}{P+W} < \tan(\alpha+\phi) < \frac{B}{P}$

we find, for stability, $\frac{B}{P+\frac{W}{2}} < \tan(\alpha+\phi)$

whence the complete condition is

$$\tan(\alpha-\phi) < \frac{B}{P+\frac{W}{2}} < \tan(\alpha+\phi) \qquad (24)$$

Note that this result depends on the assumption that the redistribution of stress when one joint tends to slide is such that the axial thrust is invariant.

VI Stress Distribution at Joint

Consider a fully elastic beam, depth 2c, subject to axial and transverse loads which produce, on a given section normal to the axis of the beam, a thrust P, a shear V and a moment M, Fig. 4 Applying the elementary theory of the distribution of stress in a beam we have, on the section under consideration,

$$\left.\begin{aligned} \sigma_x &= \frac{3My}{2c^3} - \frac{P}{2c} \\ \sigma_y &= 0 \\ \tau_{xy} &= \frac{3V}{4c^3}(c^2-y^2) \end{aligned}\right\} (25)$$

On a parallel section distant x from the first the moment becomes $M+Vx$, and we then have

$$\sigma_x = \frac{3y}{2c^3}(M+Vx) - \frac{P}{2c}$$

On an inclined section therefore, where

$$x = -y\tan\alpha$$

we have

$$\left.\begin{aligned} \sigma_x &= \frac{3M}{2c^2}z - \frac{P}{2c} - \frac{3V}{2c}z^2 \tan\alpha \\ \sigma_y &= 0 \\ \tau_{xy} &= \frac{3V}{4c}(1-z^2) \end{aligned}\right\} \quad (26)$$

$$z = \frac{y}{c}$$

The stress normal to and along the section are given by

$$\sigma = \sigma_x \cos^2\alpha + \sigma_y \sin^2\alpha + 2\tau_{xy}\sin\alpha\cos\alpha$$

$$\tau = \tau_{xy}(\cos^2\alpha - \sin^2\alpha) + (\sigma_y - \sigma_x)\sin\alpha\cos\alpha$$

whence

$$\sigma = \left(\frac{3M}{2c^2}z - \frac{P}{2c}\right)\cos^2\alpha + \frac{3V}{2c}(1-2z^2)\sin\alpha\cos\alpha$$

$$\tau = \frac{3V}{4c}(1-z^2)\cos^2\alpha - \left(\frac{3M}{2c^2}z - \frac{P}{2c}\right)\sin\alpha\cos\alpha - \frac{3V}{4c}(1-3z^2)\sin^2\alpha$$

The joint will tend to open if at any point $-1 < z < 1$ we have $\sigma > 0$. In this case

$$\left(\frac{3M}{2c^2}z - \frac{P}{2c}\right)\cos^2\alpha + \frac{3V}{2c}(1-2z^2)\sin\alpha\cos\alpha > 0$$

$$\frac{3M}{c}z - P + 3V(1-2z^2)\tan\alpha > 0$$

If $V\tan\alpha < \left|\frac{M}{4c}\right|$ this leads to

$$P < \left|\frac{3M}{c}\right| - 3V\tan\alpha \qquad (27)$$

and if $V\tan\alpha > \left|\frac{M}{4c}\right|$ to

$$P < \frac{3M^2}{8Vc^2\tan\alpha} + 3V\tan\alpha \qquad (28)$$

In the former case the joint begins to open at the edge where the moment tends to induce tension. In the latter case it first opens away from the edges, but as P is decreased (or $|M|$ increased) the opening spreads to the edge and the situation becomes identical with that of the former case.

Assume M to be positive. This may be done without loss of generality, for the sign of M may be changed by changing the positive direction of the y-axis. This also changes the signs of V and α, but the above equations continue to hold. Let

$$P = \frac{3M}{c} - 3V \tan\alpha$$

Then

$$\sigma = -\frac{3M}{2c^2}(1-z)\cos^2\alpha + \frac{3V}{c}(1-z^2)\sin\alpha\cos\alpha$$

$$\tau = \frac{3V}{4c}(1-z^2)(\cos^2\alpha - 3\sin^2\alpha) + \frac{3M}{2c^2}(1-z)\sin\alpha\cos\alpha$$

and at $z = 1$, $\sigma = \tau = 0$

Suppose

$$P < \frac{3M}{c} - 3V \tan\alpha$$

and that the joint is open for $a < z < 1$. Consider the section normal to the axis of the beam and intersecting the joint at $z = a$. Applying the elementary theory, the distribution of σ_x on this section must be linear, say

$$\sigma_x = f_0 + f_1 z$$

If we continue to take $\sigma_y = 0$ we have, at $z = a$

$$\sigma = (f_0 + f_1 a)\cos^2\alpha + 2\tau_{xy}\sin\alpha\cos\alpha$$

$$\tau = \tau_{xy}(\cos^2\alpha - \sin^2\alpha) - (f_0 + f_1 a)\sin\alpha\cos\alpha$$

Assume that at $z = a$

$$\sigma = \tau = 0$$

Then

$$(f_0 + f_1 a)\cos^2\alpha + 2\tau_{xy}\sin\alpha\cos\alpha = 0$$

$$\tau_{xy}(\cos^2\alpha - \sin^2\alpha) - (f_0 + f_1 a)\sin\alpha\cos\alpha = 0$$

and eliminating τ_{xy} from these equations

$$f_0 + f_1 a = 0$$

The total thrust on the section is P and the moment is

$M - Vac\tan\alpha.$ We therefore have

$$\int_{-c}^{ac} \sigma_x\, dy \equiv c\int_{-1}^{a} \sigma_x\, dz = -P$$

$$\int_{-1}^{a} (f_0 + f_1 z)\, dz = -\frac{P}{c}$$

$$f_0(1+a) - \tfrac{1}{2} f_1 (1-a^2) = -\frac{P}{c}$$

and $$\int_{-c}^{ac} \sigma_x y\, dy \equiv c^2\int_{-1}^{a} \sigma_x z\, dz = M - Vac\tan\alpha$$

$$\int_{-1}^{a} (f_0 + f_1 z) z\, dz = \frac{M}{c^2} - \frac{V}{c} a\tan\alpha$$

$$-\tfrac{1}{2} f_0 (1-a^2) + \tfrac{1}{3} f_1 (1+a^3) = \frac{M}{c^2} - \frac{V}{c} a\tan\alpha$$

We now have three equations in a, f_0 and f_1 which may be solved to give

$$a = \frac{\frac{M}{c} - \frac{2}{3}P}{V\tan\alpha - \frac{1}{3}P}$$

$$f_0 = -\frac{2a}{(1+a)^2}\cdot\frac{P}{c}$$

$$f_1 = \frac{2}{(1+a)^2}\cdot\frac{P}{c}$$

whence

$$\sigma_x = -\frac{2(a-z)}{(1+a)^2}\cdot\frac{P}{c}$$

and at $z = -1$

$$\sigma_x = -\frac{2}{1+a}\cdot\frac{P}{c}$$

These formulae apply in the range $1 > a > -1$ The first limit corresponds, as already shown, to

$$P = \frac{3M}{c} - 3V \tan\alpha \qquad (29)$$

and the second to

$$P = \frac{M}{c} + V \tan\alpha \qquad (30)$$

which is the value of P below which the joint will fail by complete opening.

VII NOTATION

a Proportion of length of joint or of half-depth of block.

a_1, a_2 Coefficients in formula for a.

B Compressive force across joint due to rock bolt.

c Half-depth of block.

f Function symbol

f_0, f_1 Coefficients in formula for σ_x

M Moment acting on normal section or on joint plane.

P Resultant axial thrust acting on normal section or on joint plane.

S Normal tension acting on joint plane.

T Shear force acting on joint plane

$t = \frac{y}{c}$

V Normal shear acting on normal section or on joint plane.

W Weight of block of rock between two joints

x, y Rectangular coordinates, x in direction of axis, and y normal to it.

α Angle between normal to joint plane and axis of block.

θ Angle between rock bolt and normal to axis of block.

ϕ Angle of joint friction.

ψ Angle between joint plane and joint surface or direction of sliding.

ψ_0 Maximum angle between joint plane and joint surface

μ Coefficient of joint friction or internal friction.

σ Normal stress on joint plane

σ' Normal stress on joint surface or plane of sliding.

σ_ν Normal stress in joint plane.

σ_x, σ_y Normal stresses on planes perpendicular to x--and y--axes.

τ Shear stress on joint plane.

τ_o Cohesion: shear strength with normal stress zero.

τ' Shear stress on joint surface or plane of sliding.

τ_{xy} Shear stress on planes prependicular to X - and y - axes.

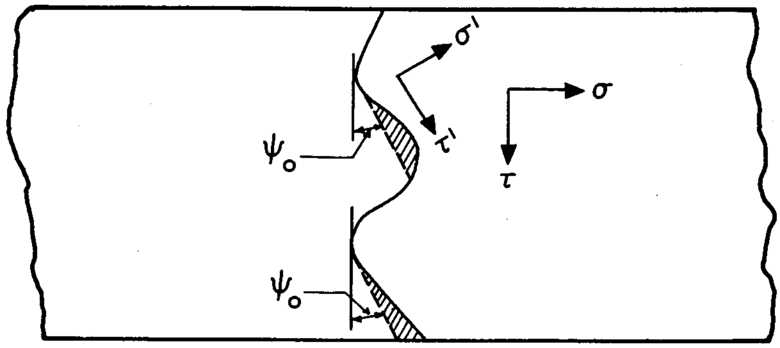
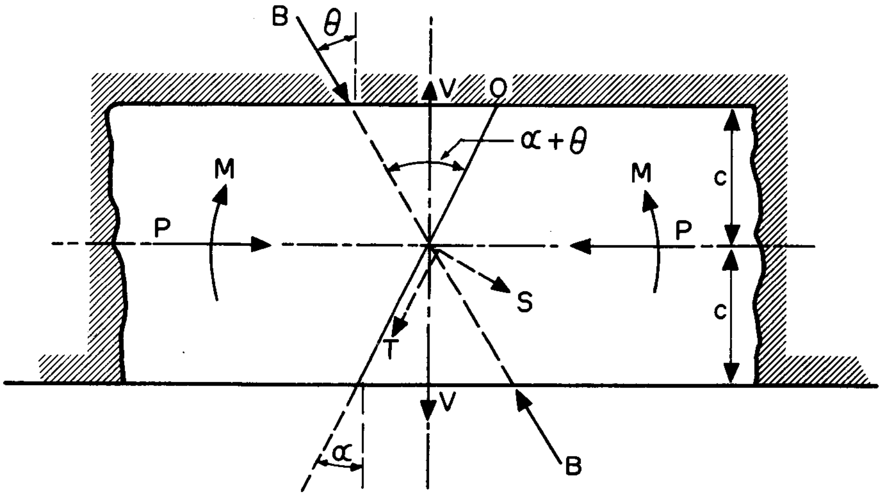
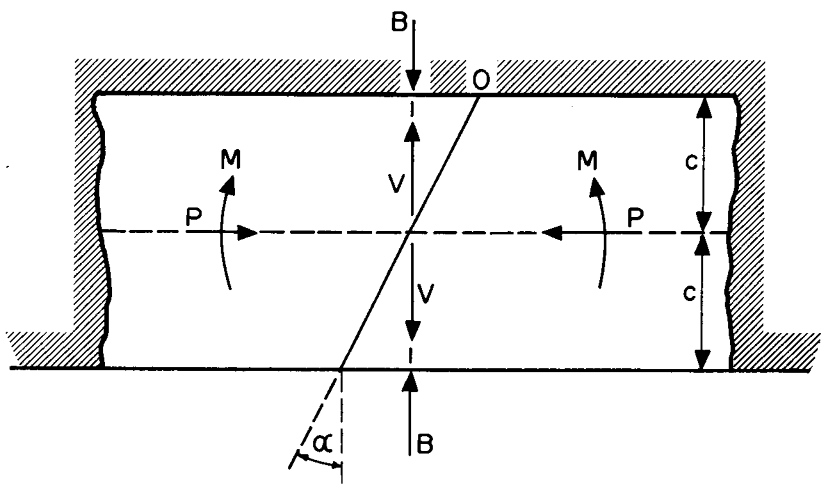


Fig. 1 — Rough joint



(a) General case



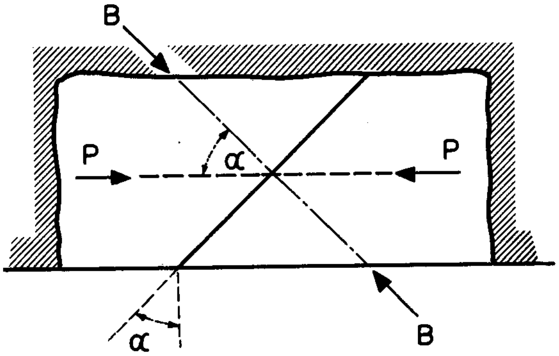
For stability:

1. Sliding: $\tan(\alpha-\phi) < \frac{B-V}{P} < \tan(\alpha+\phi)$

2. Rotation about O: $B \tan\alpha > \frac{M}{c} - P + V\tan\alpha$

(b) Case for $\theta = 0$

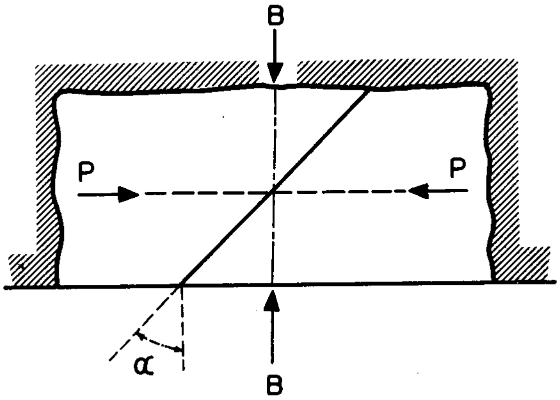
Fig. 2 — Plane joint



For stability:

$$\frac{B}{P} > \sin \alpha \;(\cot \phi - \cot \alpha)$$

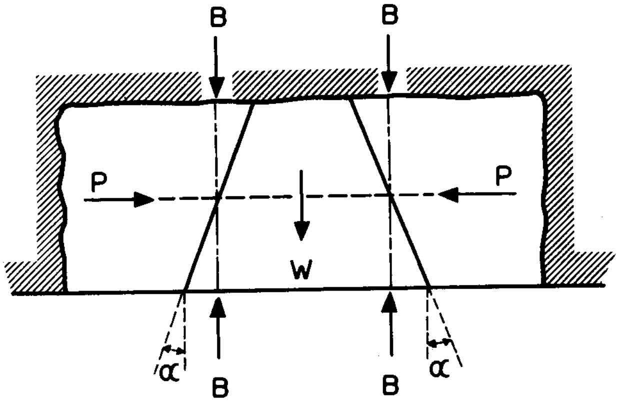
(a) Single joint with bolt normal to joint



For stability:

$$\tan(\alpha - \phi) < \frac{B}{P} < \tan(\alpha + \phi)$$

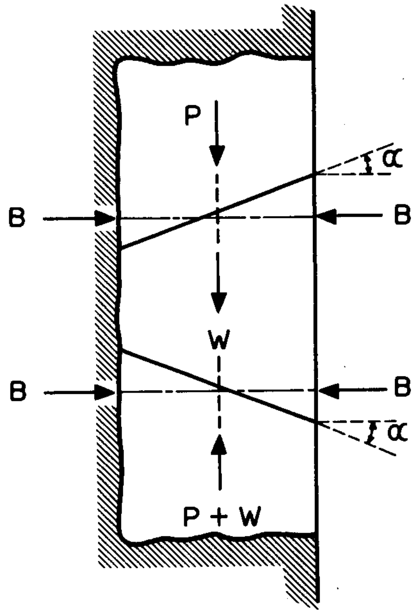
(b) Single joint with bolt normal to surface



For stability:

$$\tan(\alpha - \phi) < \frac{B - \frac{1}{2}W}{P} < \tan(\alpha + \phi)$$

(c) Block in horizontal surface



For stability:

$$\tan(\alpha - \phi) < \frac{B}{P + \frac{1}{2}W} < \tan(\alpha + \phi)$$

(d) Block in vertical surface

LEGEND:

B = Force exerted by bolt

P = Direct force on joint

$\tan \phi$ = Coefficient of joint friction

W = Weight of block

α = Angle between P and normal to joint

Fig. 3 — Simple joints

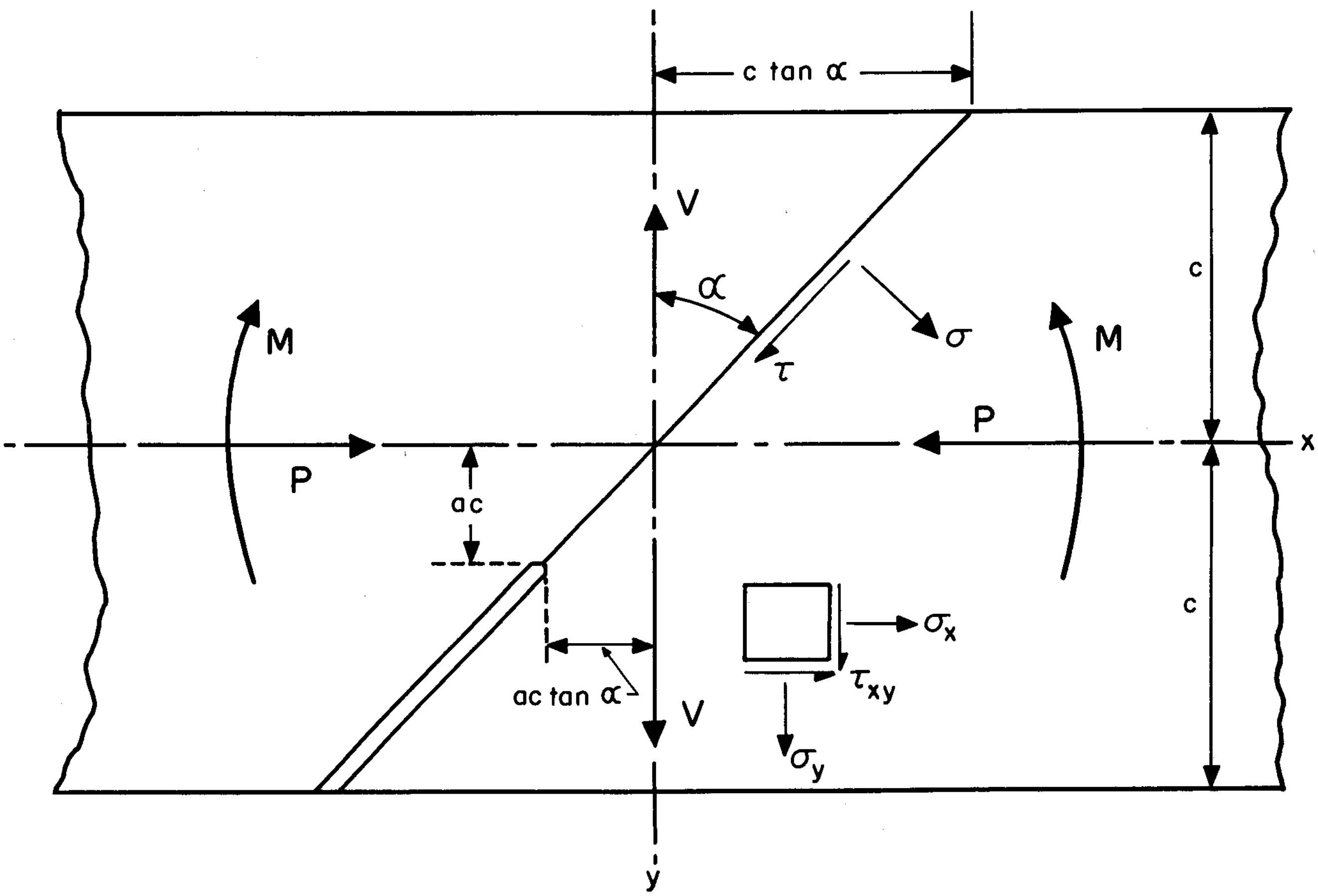


Fig. 4 — Stress distribution in joint

MAJOR UNDERGROUND EXCAVATIONS OF THE PACIFIC GAS AND ELECTRIC COMPANY

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SUMMARY

Underground excavation is a major feature in many of the Pacific Gas and Electric Company's 62 hydroelectric developments. The excavations are primarily in tunnels and surge tanks, and forty years of experience with 73 tunnels and their associated surge tanks are summarized. The successful experience with many unsupported and unlined excavations is extensive. Two of the three underground hydroelectric plants in the United States are P. G. and E. Company plants, the most recent being the Haas Plant. The Haas Plant excavation is discussed in detail. A resumé of a study of the world's underground hydroelectric plants, made in connection with the Haas Plant design, is presented.

INTRODUCTION

The P. G. and E. Company serves Central and Northern California. The electric production comprises 14 thermal electric plants and 62 hydroelectric plants having a total capacity of 5,219,000 kilowatts. The present expansion program includes conventional and nuclear thermal electric plants, hydroelectric plants, and a thermal electric plant which harnesses the natural steam of geysers. The Company looks to nuclear plants to take an increasing share of the growing load, and is engaged on five separate ventures directed toward broadening the new technology and improving the economics of future nuclear plants. Though the emphasis in present thinking is toward nuclear power, the major background of experience is in the hydroelectric field.

The purpose of this paper, as a part of The Second Protective Construction Symposium, sponsored by The RAND Corporation, is to review the successful experience with the Company's extensive hydroelectric excavations, and to present data on the excavations of the recently completed Haas underground hydroelectric plant.

TUNNELS

The hydro plants are nearly all located in the Sierra Nevada Mountain Range which extends some 430 miles along the eastern boundary of California. Except in the northern end of the Sierras, where the mountains are of volcanic origin, the basic formation is granitic. At the lower elevations is the Calaveras formation. In the steep California Sierra Nevada Mountains, the typical hydroelectric river development consists of high-elevation storage reservoirs at 5000 to 8000 ft elevation, a chain of plants, each consisting of diversion dam, pressure tunnel, penstock and powerhouse; and finally a large multiple-purpose reservoir at the toe of the mountains and beginning of the valley floor. P. G. and E. Company has 62 plants which have 73 tunnels totaling 119 miles in length. The tunnels are generally of horseshoe shape and range in size from 8 to 26 ft diameter. Figure 1 presents data on the more important tunnels. They are tabulated in order of date completed. All the tunnels are still in service, and the operating experience covers a period of 40 years.

It may be noted in Fig. 1 that about one-half the total length of tunnels is unlined. An unsupported concrete-lined tunnel costs roughly 50 per cent more than an unlined tunnel of equal hydraulic head loss.

Consequently, where the rock is suitable, the Company's tunnels are not lined. The unlined tunnel, because of its greater hydraulic roughness, is larger in diameter than its hydraulically equivalent concrete-lined tunnel. The experience with the concrete-lined tunnel sections has been trouble-free, except for instances where concrete in the earlier years was not poured to today's standards, and some repairs have been necessary. The unlined tunnel experience has been exceptionally good and the amount of unlined tunnel experience may be expressed as 950 mile-years of service. No damage due to earthquakes has occurred.

The power pressure tunnels are subject to sudden changes of pressure due to changes in load at the power house. Normal operating pressure changes of 5 to 15 psi may occur instantaneously, and of 25 to 40 psi in a minute or several minutes. The "instantaneous" pressure changes are from waterhammer pressure waves that travel at about 4600 ft/sec, and the slower changes are from the varying water level in the surge tank. Upon a rapid pressure drop in the unlined tunnel, the water in seams or cracks near the tunnel wall may be visualized as exerting a pressure tending to spall the rock. Also, water may erode seams in jointed rocks, causing rockfalls. To date, only four rockfalls have occurred which were of sufficient magnitude to require maintenance. The shutdowns for the work were not emergency but scheduled.

At the Colgate tunnel on the Yuba River, two falls occurred which opened holes 16 ft above the crown, one 10 ft and one 20 ft long. These falls were the result of saturation and softening of serpentine kidneys or lenses. Repair was carried out by installing timber sets, and cribbing, and guniting between and over the sets. At the Balch plant on the Kings River, the tunnel was unwatered after the first twelve years of operation for its first routine inspection and two rock falls were found. Water had eroded soft seams to a depth of several feet and adjacent rock fell into the tunnel. Repair work consisted of filling the seams with gunite. The Poe Plant on the Feather River had one similar fall and repair. At Kern Canyon, on the Kern River, a flat roof tunnel with horizontal bedding planes had enlarged in a faulted zone due to progressive rockfalls. The tunnel had enlarged itself in width and height from 13 ft to 20 ft for a length of about 30 ft. In this instance the trouble was observed in 1950 and repaired in 1958 during a power house outage for turbine overhaul. Repair was by rock anchors and gunite. In every case, the rock falls were not serious but would have become progressively worse with time.

For a power tunnel there is little choice of location to gain improved rock conditions, and tunnels in poor rock are sometimes unavoidable. Every attempt is made to obtain the best location, since supported and lined tunnel costs a little more than twice as much as unlined tunnel. In the fractured and changeable volcanic formations, the tunnels are usually 100 per cent supported and lined, whereas the granite tunnels are generally 90 to 100 per cent unlined. There is an economic compensation for this in that the porous volcanic watershed provides free storage, and a more uniform runoff throughout the year than the granite watersheds. In the Calaveras formations of greenstone, serpentine, schist and slate, it is difficult to predict how much will require support and lining. In general, the economy in the use of unlined tunnel is so great that the practice is to leave it unlined if it does not require support.

The comments on unlined pressure power tunnels are considered to be of value to the purposes of this symposium since they show the satisfactory service of unlined rock excavations over a long period of time under adverse conditions.

Cost. The cost per foot of tunnel of any one size varies with location, rock conditions, length, the amount of construction activity throughout the country at the time the tunnel is bid, and other factors. Tunnel costs, of course, have been rising along with the general cost increases in other types of construction. For cost estimating of a particular tunnel, knowledge of the geology is essential. Also, either considerable recent experience or the advice of a construction engineering consultant is necessary. However, it is useful to know the relative cost per cubic yard of excavation for tunnels of different sizes in preliminary studies, in order to arrive at the economic size. Figure 2 is presented with this limited purpose in mind.

Figure 2 gives an estimated cost per cubic yard for unsupported horseshoe-shaped tunnel of a length of one to several miles driven from one heading. It is observed that the cost per cubic yard decreases materially as tunnel size increases in diameter from 8 to 20 ft and levels off as tunnel size goes beyond 20 ft diameter. As the tunnel increases in size in the 8 to 20 ft range, the decreasing cost per cubic yard is due in large part to: costs of tracks, air supply and supervision increasing only moderately; improved working space and working conditions; use of larger equipment; reduction in drilling and powder per cubic yard; and pulling of longer "rounds." For supported tunnel, the driving costs of Fig. 2 are higher by perhaps 25 to 60 per cent due not only to the cost of the supports, but to the lower daily progress in the supported tunnel while the payroll and overheads remain the same.

SURGE TANKS

At the lower end of the power pressure tunnels a surge tank is generally necessary to supply or receive water due to rapid load changes at the power house, while the flow in the tunnel takes the necessary time to accelerate or decelerate. Four surge tanks are shown on Fig. 3. In the case of surge tanks, there is an opportunity to reduce costs and risks in excavation by finding the most favorable rock conditions near the portal, and this accounts for the different layouts shown in Fig. 3.

At the Rock Creek Plant on the Feather River, there was deep decomposed granite along the only ridge available for the tunnel portal, and a railroad and highway were located below. Consequently, the surge tank was moved back into the mountain where the presence of good granite rock was confirmed by the tunnel below and the access adit above. At the Cresta Plant just downstream from Rock Creek, the tunnel portal ridge consisted of deep decomposed granite. An underground shaft surge chamber with the necessary open cut would have presented hazards and high costs. The surge tank therefore was moved to a nearby solid granite creek bed and connected by a lateral tunnel in good granite underlying the decomposed granite ridge. At the Pit 4 and Poe plants, the rock conditions are uniformly poor in the general area of the portal, and the surge tanks were located to provide the most favorable open cut. The tunnels were run directly under them.

The usual construction method is to drill a 6 to 8-in. hole on the centerline of surge tank; construct a raise of 8 to 10-ft dimension from a cage suspended by cable through the drilled hole; enlarge the raise by benching down in 8 to 10-ft steps and dropping the muck into the raise; and remove muck from a hopper at base of the raise. At Rock Creek and Cresta, the excavations were in granite and required no support. At Pit 4 and Poe, in fractured volcanic formations, latticed steel ribs and lagging were necessary. Ribs and lagging were embedded in the concrete lining.

The recently-constructed Poe surge tank excavation was begun as a 7 x 14-ft raise with manway. After 55 ft of progress, the rock conditions became unsuitable for raise construction and the muck shaft was completed from the top. The tank was then benched down in steps, using the muck shaft for removal of material. Concrete was placed using slip forms and the 212-ft shaft and riser (Fig. 3) was poured in 250 hours of continuous work at 9.6 in. per hour. The slip form method was developed in grain elevator construction, and very successfully used in this case by Utah Construction Company.

WORLD'S UNDERGROUND HYDROELECTRIC PLANTS

In planning and designing the Haas underground plant, a thorough review of literature on the world's underground hydroelectric plants was made. The study was furthered by work of the Power Division Technical Committee of the American Society of Civil Engineers and by correspondence with foreign engineers. Resulting from this is Ref. 1 which presents a selected bibliography of 213 references with abstracts, and a list of the world's underground hydroelectric plants with data including the size of excavations. The excavations are large, some typical sizes, in length x width x height being: Darquinah - 239 x 106 x 85 ft; T-1, 305 x 59 x 105 ft; Kitimat (ultimate) 1140 x 82 x 139 ft; and Kariba, 2 - 460 x 75 x 125 ft.

Reference 1 was the first of a group of papers presented at a Symposium on Underground Hydroelectric Plants sponsored by the American Society of Civil Engineers at the Annual Meeting in New York in October 1957. These papers may in the future be assembled in a single Symposium volume. At present, they are available as ASCE Proceedings Separates:

Number	Subject	Author
1350	Bibliography and List	J. B. Cooke & A. G. Strassburger
1529	Haas Plant	J. B. Cooke
1554	Plants in Italy and Other Countries	Claudio Marcello
1555	Sudagai Plant, Japan	Tatsuo Mizukoshi
1598	Ambuklao Plant, Philippines	Andrew Eberhardt
1670	Plants in Canada	A.W.F. McQueen, C. N. Simpson, and I. W. McCaig
1675	Plants in Scotland	C. M. Roberts

Number	Subject
1460	Discussion of 1350
1538	Discussion of 1350
1689	Discussion of 1529, 1554 and 1555
1830	Discussion of 1350, 1598
1953	Discussion of 1529, 1554, 1555, 1598, 1670 and 1675

Since the underground excavation is a major feature and cost item in the underground plant, many of the references in the bibliography and the Symposium papers discuss the exploration, design and construction of the excavation itself.

Reference 1 lists 296 underground hydroelectric plants as existing under construction or scheduled. The strong world trend toward the underground construction is indicated by the fact that only 40 plants existed before 1946. The basic reason for adopting the underground plants has been economic. Many underground plants have cost less than the alternative surface scheme; or, with credit for the additional effective head developed, they produce power at the lowest unit cost. Others are considered economic when credit is given to scenic benefits, freedom from slides, avalanches or freezing, or other semi-tangible benefits. There seems to be a reluctance to acknowledge the extent to which national security is a consideration.

The following tabulation gives a distribution of the plants by countries:

Algeria	2	France	19	Philippines	1
Australia	9	Germany	3	Portugal	5
Austria	2	Guinea (French)	2	Rhodesia	2
Brazil	4	Iceland	1	Scotland	5
Canada	3	India	3	Spain	2
China (Formosa)	3	Italy	72	Sweden	44
Cuba	1	Japan	5	Switzerland	28
Egypt	1	Mexico	4	United States	4
El Salvador	1	Norway	53	USSR	3
Finland	2	Peru	1	Yugoslavia	12

The list is rather complete except in the USSR and the Balkan countries. That there are so few underground plants in the United States raises the question, why? First, the reasons of weather, avalanches, severe freezing, scenic considerations, and national security have not been important in this country. From the basic economic standpoint, in foreign countries the underground scheme is relatively more economic than its surface alternative. The underground scheme requires more labor and less material. Labor is a relatively more important cost item in the United States. However, advances in underground excavation techniques, equipment and knowledge are tending to increase the economic use of underground excavation in this country. The reasons for adopting an underground installation at the Company's Haas Plant on the Kings River are economic.

THE HAAS UNDERGROUND EXCAVATIONS

The Haas Power Project is one of three P. G. and E. Company projects on the North Fork of the Kings River in California, 50 miles from Fresno. It is the first large underground plant in the United States. The two other plants are small but are notable in that they are two of the first six such plants in the world. They are Snoqualmie Falls, Washington (1899) and P. G. and E.'s Spaulding No. 1, California (1917).

A general characteristic of the Haas Project is the extent of underground construction in the massive granite formation of the Sierras and the use of the granite rock as construction material. Rock excavations for the Haas power tunnel include the Wishon Dam diversion tunnel, the outlet from the dam, Haas intake valve chamber and access shaft, a 6-1/4 mile power tunnel, a rock trap, and a surge shaft with galleries. The power plant excavation contract included the tailrace tunnel, the penstock shaft, the power house access shaft, and the power house chamber. The general dimensions and arrangement of these excavations are shown on Fig. 4. In addition to taking advantage of the unlined rock excavations, granite was used for two 300-ft high rockfill storage dams, and crushed granite from the powerhouse excavation was used for making concrete.

HAAS POWER TUNNEL EXCAVATIONS

Dam Outlet and Haas Intake. The 1100-ft long diversion tunnel required no support. Except for the concrete plugs and valve discharge chamber, it all remained unlined as the permanent outlet works for the dam (Fig. 4.) The massive granite is considered capable of withstanding the turbulent discharge of 3500 cfs without concrete lining. The Haas power tunnel begins at the valve chamber where the water under 240 ft of head is sealed off from the valve chamber by well-grouted 20-ft long concrete plugs, through which a 10-ft diameter pipe carries the flow. The valve chamber and the 6 x 6 ft access and air vent shaft are unlined.

Power Tunnel Alignment and Exploration. The power tunnel is a 13-ft unlined horseshoe section, 6-1/4 miles long with a head of 240 ft at the upstream end and 330 ft at the downstream end. The alignment parallels the canyon wall. Locating the tunnel further into the mountain than necessary to assure favorable granite for driving, and to assure adequate cover to prevent blowout and minimize leakage, increases the length unnecessarily. To aid in establishing the alignment, drill holes were located in four creek beds that would control the alignment. The top of the holes was at elevation 6550, the maximum Wishon Reservoir level, and they were drilled to tunnel grade. It was considered that a tunnel alignment with cover equal to static head would be accepted if the lower half of the holes showed sound unweathered granite. Two of the holes showed decomposed and weathered granite in the upper half of the hole and two showed 100 per cent to be sound fine-grained granite, and the alignment was accepted. Lateral cover at these creeks is about three times static head. An exception to the criteria of minimum cover equal to static head was made at one creek where surface granite was massive and no drill hole was put down. At this creek 180 ft of cover over the unlined tunnel was accepted where the head in the tunnel was 280 ft, in order to save several hundred feet of tunnel length. An accurate leakage test made four days after filling the tunnel at maximum head indicated 0.6 cfs leakage in the 6-1/4 mile unlined tunnel under 240 to 330 ft of head. Inflow leakage during construction did not exceed 0.7 cfs at any time.

Power Tunnel Construction. The tunnel excavation work was awarded on the basis of competitive bidding to Morrison-Kaiser-Macco-Perini, with Morrison-Knudsen Company as sponsor. The 32,850-ft long tunnel was driven from one adit on two headings; 19,560 ft upstream, and 12,830 ft downstream. The first 2800 ft in each direction from the adit was driven by the method of "alternate headings" at an average rate of 90 ft per day, for one crew on each of three shifts, until daily progress dropped to the required minimum schedule of 1000 ft per month (40 ft per day) in the long upstream heading. The overall average rate of driving was 57 ft per day, including the slower alternate heading driving. Maximum single heading progress was 70 to 75 ft per day and monthly averages in the latter half of the job were 65 to 70 ft per day. Construction is further covered in Refs. 3, 4 and 5. The average overbreak was 21.9 per cent, which is equivalent to an overbreak of 0.71 ft all around the 13-ft horseshoe design section. The tunnel is 97 per cent unlined with 14 short pieces of concrete or gunite lining of 10 to 150-ft length totalling 1020 ft in faulted, soft seam and fractured zones that required support. Also, gunite was used on occasional seams that might otherwise erode and cause rockfalls.

Rock Trap. After the rails and ties were removed from the floor of the unlined tunnel, 12 to 15 in. of tunnel muck remained. To prevent this material from reaching the turbines, a trap is located at the downstream end of the tunnel. This trap (Fig. 4) is an enlargement of the 13-ft tunnel to a 21-ft unlined tunnel for a length of 440 ft. It will store the portion of the muck that is estimated to move down the tunnel. The arrangement is substantially lower in cost than removing the muck or concreting the invert.

Surge Chamber. Two drill holes indicated that the 300-ft deep surge shaft would have to be located with the upper one-half in decomposed and weathered soft-grained granite. The drill holes also indicated that the upper formation was not suitable for a "raise" and that it would be safer to excavate downward. Accordingly, the contractor excavated and concreted the top 150 ft from a head frame at the surface. The procedure was to excavate 10 ft and then concrete-line it. When good rock was encountered 150 ft down, a 6-in. hole was drilled to the tunnel. The lower 150 ft was then excavated from the bottom up, from a cage suspended on a cable in the 6-in. drill hole. The lower 150 ft of the 12-ft diameter shaft was gunited, 3 in. thick with 4 x 4 by #6 mesh, since the granite was water-bearing and seamy. The two 75-ft long by 12-ft horseshoe-shaped galleries were left unlined. From the surge chamber to the portal the tunnel is designed for pressure and watertightness, having a concrete backfilled steel liner.

THE HAAS UNDERGROUND POWERHOUSE

Decision to Go Underground. The Haas Plant was originally laid out as a surface scheme and on essentially the same alignment as the adopted underground scheme. The surface layout was the most economic of five alternative surface schemes, but the high cost and head loss of the heavy-walled penstock (2 to 3 in. thickness) on the flat profile at the lower end was discouraging. A study of an underground layout indicated that it would be substantially lower in cost and would develop 30 ft more effective head. Even if the chamber excavation were to cost several times the estimate, it was still economic to go underground. With such a margin for contingencies, and with nominal exploration, it was decided to go underground for the purpose of effecting capital savings and developing more power. Additional inherent benefits are: development of maximum effective head, conservation of 2500 tons of steel, a safer and more permanent installation, and minimum maintenance. The plant is now in operation and appreciable savings were realized as the result of adoption of the underground scheme.

Description. The underground powerhouse is approximately 500 ft vertically below the surface and 2000 ft from the river. The general arrangement and details are shown on Figs. 4 and 7. The penstock is in a 760-ft shaft leading to the powerhouse chamber. The chamber, which houses valves, units, auxiliaries and control room is 173 by 56 ft in plan and 100 ft high. An 18-ft diameter access shaft leading to the surface contains generator leads, elevator, stairway, piping and control leads and serves as the exhaust ventilating duct. The 2000-ft long, 17.5-ft high by 15-ft wide unlined tailrace tunnel served as access during construction and provides for future access for heavy loads, as well as for emergency personnel access and for ventilation. The Haas power development is discussed in greater detail in Ref. 2.

HAAS POWERHOUSE EXCAVATION

Exploration. The optimum economic location of the underground chamber was determined and it was decided to put down an NX core hole, 500 ft deep, at each end of the proposed chamber. The first hole DH #1, provided essentially all 10-ft cores of ideal granite. The second, DH #2, 200 ft away, was a perfect hole until the elevation of the chamber was reached, where some 50 ft of close cleavage planes and soft seams, in a plane 20° to the horizontal, were encountered. DH #2 was getting away from the ridge and toward a nearby creek which may have been in a faulted zone. DH #3 was started a distance equal to chamber length, 170 ft, from DH #1 and was essentially a perfect hole and of the same grain structure granite as DH #1. The chamber was then located between DH #1 and #3, and it was decided to go underground without further drilling.

The excavation contract was awarded to Morrison-Kaiser-Macco-Perini, sponsored by Morrison-Knudsen Company.

General. The power plant excavation contract included tailrace tunnel, powerhouse chamber, access shaft and penstock shaft. All the excavation proceeded as planned with no contingencies arising. Practically no support was required and no water was encountered. Figure 5 is a record of the excavation progress and Fig. 6 illustrates the excavation methods. Details of the excavation have been well covered in the engineering literature (Refs. 3, 4, 5, 6).

The first step in the excavation contract was to complete the tailrace tunnel in order to gain access to the power house and to the other excavations. A small chamber was excavated, from which work was begun simultaneously on the penstock tunnel and shaft, two 8 x 8-ft raises to the crown of the chamber, and a 5 x 10-ft raise of 51° to the horizontal to the base of the access shaft (Fig. 6). As indicated by Fig. 5, the excavations proceeded together, with the chamber excavation determining the completion date.

Tailrace Tunnel. The 2000-ft long tailrace tunnel (Fig. 4) was driven in 58 working days at an average rate of 34 ft per working day, which includes some initial one- and two-shift days. For three-shift driving, the average progress was 47 ft per day with maximum days of 55 to 60 ft per day. Thirteen-foot rounds were pulled and mucking was by Eimco mucker and Dumptors (Refs. 3, 6). No support was required and no water was encountered. Four 40-ft long passing areas were constructed, each giving a total width of 20 ft for passing instead of the tunnel's 15-ft width. The 15-ft width of tailrace tunnel was adopted as a practical minimum for construction loads and traffic, and is larger than would have been necessary for hydraulic purposes. The invert is paved for use as a permanent access roadway.

Rock Anchor and Gunite Arch for Power House Chamber. It was considered that, other than a few rock anchors, no actual support would be required. However, tension cracks and possible spalling in the arch ceiling could be expected. It therefore was decided on the basis of judgment to secure the ceiling for personnel and equipment safety. A rock-anchored and gunited arch was estimated to cost one-half as much as a concrete arch and to have other advantages. It could be accomplished with minimum delay and cost to the excavation, and the rough surface is desirable for accoustical reasons. Details of the arch are shown in Fig. 7. The rock anchors are 1-in. diameter reinforcing bars, 10.0, 12.5, and 15 ft long, and at 3-1/2 ft spacing both ways. The anchors and gunite in effect provide a reinforced or sewn rock arch, from which a loosened rock cannot fall. The Perfo Method of the Sika Chemical Corporation was selected as the most economic and suitable method for overhead grouting of rock anchors, and is illustrated and described in Ref. 6. Two half-round perforated tubes of sheet metal were filled with a stiff sand cement grout and then wired together. This was inserted in the hole and a reinforcing rod was driven in by an air hammer. The volume of grout was such that a slight amount was extruded as the rod was driven to the end of the hole. 4 x 4 in. x #6 x #6 gunite mesh was formed to cover the rock with 1 in. minimum clearance over rock. Two #4 by 2-ft reinforcing rods were welded to the anchor at 90° to each other and each parallel to the rock face. These bars supplement the mesh and anchor the gunite to the rock. Gunite of 4 in. thickness, with 1-1/2 in. minimum cover over mesh and rock points was applied to the shape of the rock.

Powerhouse Chamber. The size of the initial working chamber, 25 x 70 ft in plan and 23 ft high, was governed by the shape of the wheel pit and sump area which was at tailrace tunnel grade (Fig. 6). Two 8 x 8-ft raises were driven to the crown and an 8 x 20-ft crown drift was driven the full 173-ft length of the power house. In order not to interfere with the access shaft excavation, the excavation of the arch, rock anchoring, and guniting (construction steps 3, 4 and 5 of Fig. 6) were started at the opposite end. The rock anchor and gunite work was carried out from pipe scaffolding on the floor of step 5. Scaffolding was moved as work progressed toward the access shaft end of the power house. The work was well organized and the 2000 anchors and 13,000 square feet of gunite was completed in six weeks along with some remaining step 3 and 5 excavation (Fig. 6). After the arch was completed, the excavation proceeded rapidly (Fig. 5) at an average of 2000 cu yds per week with daily production varying between 200 and 1100 cu yds per day. The main excavation was carried out in irregular benches about 10 ft or more in height. Muck was brought to the chutes by a slusher which is a "bucket" operated by cables between a dead-man anchor and a compressed air hoist. The excavation between the two 8-ft square chutes was completed first in order not to interfere with the access shaft excavation. Only a few anchors were used other than the pattern (Fig. 7) established for the roof.

No water was encountered, but there were several damp spots on one wall (Fig. 8) and at each end. The moist seams were drilled to drain them and minimize dampness, but no curtain walls were used. The rock temperature was 58° F when the tailrace tunnel reached the chamber in February 1957, which is a very favorable temperature. It increased about 0.5° per month due to the warm ventilating air, and in September 1957 was 62° F.

Access Shaft. The access shaft excavation (Fig. 6) began with a 6-in. diameter hole. An 8 x 9-ft raise was driven using a cable-supported steel cage. The cage was pulled under the chamber arch and the cable raised in the 6-in. hole when a round was to be shot. The 18-ft diameter was then ringed out from the top down. The top 20 ft of the shaft was in weathered rock and was excavated from the surface bench and concreted before ringing out the 18-ft unlined shaft, which is in excellent and dry granite rock. Since the concreted rock carried water, the formation was grouted. The 389 ft of 8 x 9 ft raise was driven in 25 working days at 16 ft per day. Progress increased from 8 to 15 ft initially to 20 to 25 ft per day near completion. Ringing out took 23 days at an average of 17 ft per day.

Penstock Shaft. After 635 ft of the 760-ft shaft was raised, fractured and then soft-grained granite was encountered. The remaining 125 ft was driven from the surface bench downward and required timber support for safety. Men and supplies were brought to the raise heading by a small car in the manway (Fig. 6). The car was pulled by a tugger hoist located in the 9-ft penstock branch excavation (Fig. 6). The muck was moved by rail through the 12-ft branch excavation and dumped into the chamber. The branch pipes are only 4 ft-7-1/2 in. OD, but the 12-ft size tunnel is necessary to install the two 82-in. diameter bend pieces and the wye. The raise was driven in 113 days at an average rate of 7 ft per day, with days varying from 5 to 10 ft depending on whether one or two rounds were pulled.

CONCLUSION

The extensive experience with tunnels and surge tanks has been very satisfactory. The Haas Plant went into operation in December 1958. There has been no instance of a falling rock from the access shaft or the unlined powerhouse chamber walls.

In looking to the future, it is possible that nuclear plants may involve greater excavations than heretofore. The Pacific Gas and Electric Company has underway with General Electric Company a research and development project on the "pressure suppression" method of reactor containment. This project has received encouragement from the United States Atomic Energy Commission, and so far the investigations and experimental results have been gratifying. Success of the program could result in substantial economies and could eliminate from the water-cooled nuclear plant the familiar sphere or cylindrical capsule which has been a trademark of most atomic stations. One concept of a plant using "pressure suppression" (Fig. 10) would have the reactor housed in a small-diameter steel-lined caisson about 100 ft deep. Pipes would connect to an underground tank containing water and air, and into which steam would escape from the dry well reactor enclosure in the event of an unusual incident. Aside from the sub-surface arrangement that may develop with "pressure suppression" containment, consideration is given to possible future underground nuclear plants (Ref. 7).

REFERENCES

1. J. Barry Cooke and Arthur G. Strassburger, "Bibliography: Underground Hydroelectric Plants," Proceedings of the American Society of Civil Engineers, Paper 1350, August, 1957. Discussion in Papers 1460 and 1538, and authors' closure in Paper 1830, October 1958.

2. J. Barry Cooke, "The Haas Hydroelectric Power Project," Proceedings of the American Society of Civil Engineers, Paper 1529, February, 1958. Discussion in Paper 1689 and authors' closure in Paper 1953, February, 1959.

3. "Haas Underground Powerhouse," Pacific Road Builder and Engineering Review, V. 88, June, 1957, pp 25-33.

4. "Kings River Construction; Haas Powerhouse, Tunnel," The EM-Kayan, J-16, The Morrison-Knudsen Company, Boise, Idaho, August, 1957, pp 3-5.

5. L. L. Wise, "P. G. and E. Goes Underground for Power," Engineering News Record, V. 159, September 19, 1957, pp 51-54.

6. "Crews Hollow Mountain to Build Underground Power House," Construction Methods and Equipment, V. 39, Sept. 1957, pp 166-175.

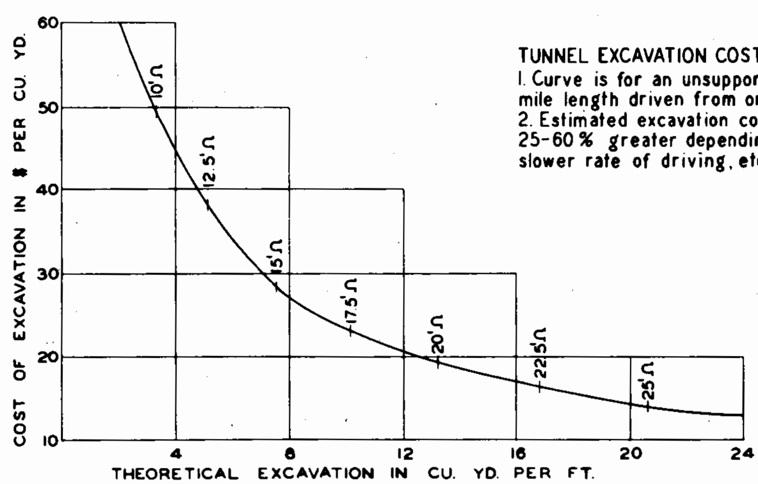
7. Christian Beck, Engineering Study on Underground Construction of Nuclear Power Reactors, United States Atomic Energy Commission, AECU-3779, April 15, 1958.

FIGURES

PHYSICAL DATA — P.G.&E. CO. TUNNELS								
The P.G.&E. system contains 73 tunnels totaling 119 miles in length. This tabulation includes 32 of the more important tunnels totaling 93.3 miles in length.								
NAME OF TUNNEL	YEAR COM-PLETED	LENGTH (FEET)	EXCAVATED CROSS SECTION W x H (FEET)	LINED CROSS SECTION W x H (FEET)	PERCENTAGE			TYPE OF ROCK
					SUP-PORTED	LINED	UN-LINED	
KERCKHOFF	1920	18,900	18 x 18	——	0	0	100	GRANITE
KERN	1921	8,350	13 x 14	11.25 x 12.17	20	56	44	GRANITE
PIT 1	1922	10,050	15.75 x 14.5	14.25 x 13 *H	100	100	0	VOLCANIC
BALCH	1924	19,000	12 x 12	——	0	0	100	GRANITE
PIT 3	1925	20,981	22 x 21.5	19 **C	98.7	100	0	VOLCANIC
BUCKS CREEK NO. 1	1925	9,550	9.25 x 7.5	——	0	0	100	GRANITE
BUCKS DIVERSION	1925	5,750	8 x 6.75	——	0	0	100	GRANITE
MELONES	1927	4,960	13.5 x 13.5	12 x 12 HC	0	100	0	GREENSTONE
DRUM CANAL NO. 1	1928	3,350	9.87 x 13.67	——	0	0	100	GRANITE
TIGER CONDUIT	1932	14,353	10 x 11.25	7.67 x 9.25 H	6.3	6.3	93.7	GRANITE
STANISLAUS	1939	57,509	9.5 x 11.25	9 x 10.75 H	1.3	1.4	98.6	CALAVERAS
COLGATE	1940	24,674	9.5 x 10.75	8 C	3.0	7.2	92.8	GREENSTONE & SERPENTINE
OLEUM	1942	2,576	9.75 x 11	9 x 10.25 H	100	100	0	SANDSTONE
DUTCH FLAT	1943	21,775	12 x 12	9 x 9 H	59.2	64.6	35.4	SLATE-SERPENTINE
NARROWS	1943	1,058	10.5 x 13.5	9 x 12 H	0	100	0	DIABASE
PIT 5 NO. 1	1944	5,037	21	19 C	100	100	0	VOLCANIC BRECCIA
PIT 5 NO. 2	1944	23,161	21 x 21	19 x 19 HC	100	100	0	VOLCANIC BRECCIA
TABEAUD	1948	2,889	13.5 x 13.5	12 x 12 H	0	100	0	CALAVERAS SCHIST
WEST POINT	1948	14,333	12.83 x 14.5	10.83 x 13.5 H	6.4	11.5	88.5	GRANITE
ELECTRA	1949	43,062	12.83 x 15.5	10.83 x 13.5 H	21.7	21.7	78.3	GRANITE & CALAVERAS
CRESTA	1949	20,903	26 x 25	24 x 24 H	8	9.6	90.4	GRANITE
ROCK CREEK	1950	34,119	25 x 25	21.87 x 21.67 H	61.8	64.8	35.2	GRANITE
BEAR RIVER DAM	1952	1,095	10 x 12	——	0	0	100	GRANITE
BEAR RIVER	1952	13,249	8.5 x 11	7.5 x 9 H	0.5	2.9	97.1	GRANITE
HENDRICKS	1953	4,820	7.5 x 8	5 x 6.5 H	100	100	0	VOLCANIC-CONGLOMERATE
PIT 4	1954	21,454	20.5 x 20.5	19 x 19 HC	100	100	0	VOLCANIC-TUFF-BRECCIA
WISHON	1958	1,158	15 x 15 & 15 x 21	——	0	0	100	GRANITE
COURTRIGHT	1956	540	8.5 x 8.5	——	0	0	100	GRANITE
		675	10 x 10.25	——	0	0	100	
POE	1958	32,834	24 x 24	19 x 19 HC	91.8	93.4	6.6	SERPENTINE & DIORITE
BUTT VALLEY	1958	10,899	14.5 14.5	13 HC	100	100	0	SHALE-VOLCANIC-CLAY
CARIBOU NO. 2	1958	8,710	14.5 14.5	13 HC	100	100	0	SERPENTINE & SHALE
HAAS	1958	32,854	13 x 13	11 x 11 H	3	3	97	GRANITE

*H-HORSESHOE SECTION **C-CIRCULAR SECTION

FIG. 1 PHYSICAL DATA - P. G. AND E. CO. TUNNELS



TUNNEL EXCAVATION COST
1. Curve is for an unsupported tunnel of approximately 1 to 3 mile length driven from one heading.
2. Estimated excavation cost of supported tunnel is some 25-60% greater depending on predicted geology, set spacing, slower rate of driving, etc.

FIG. 2 EXCAVATION COST - UNSUPPORTED TUNNELS

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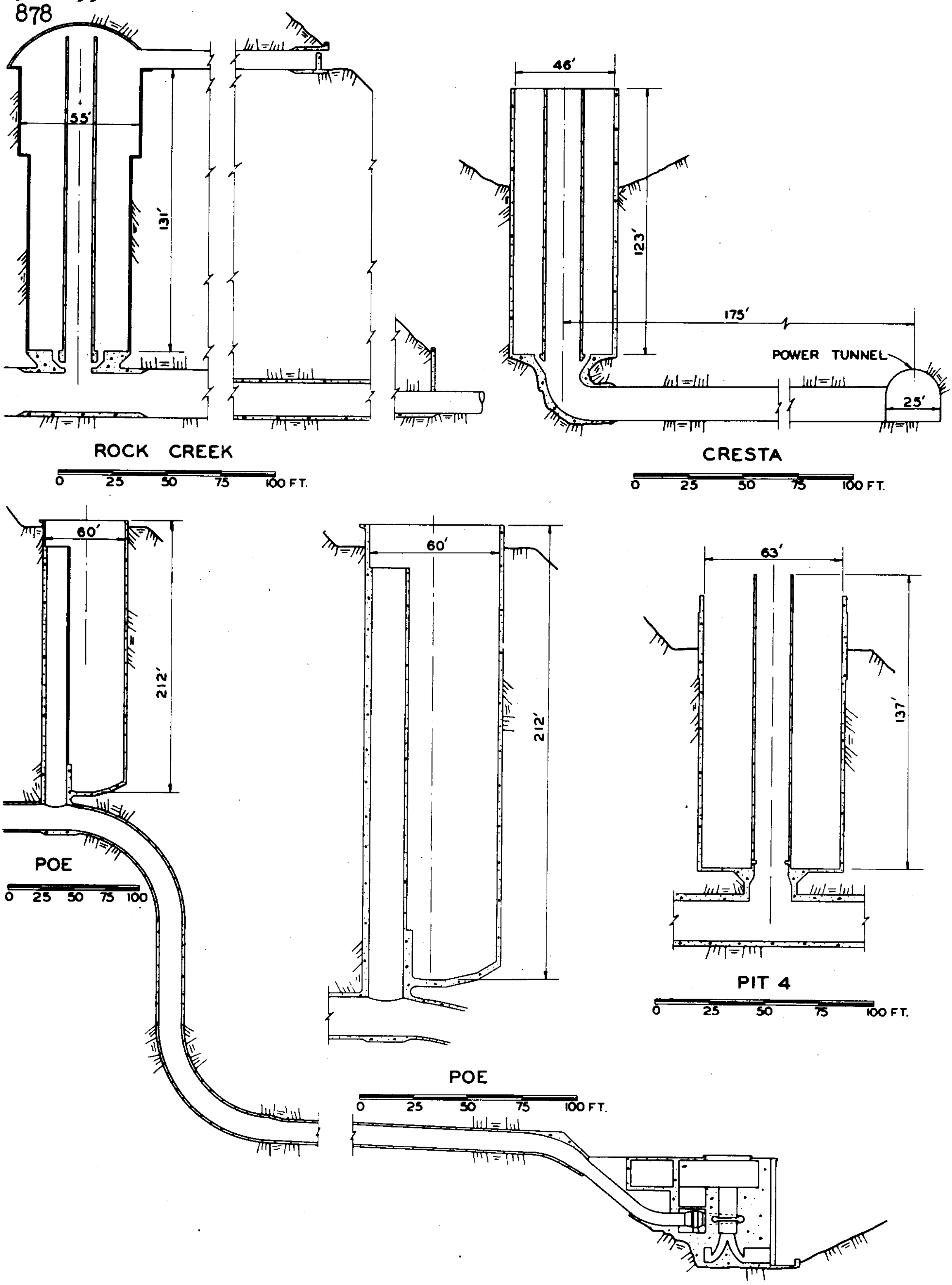


FIG. 3 ROCK CREEK, CRESTA, PIT 4, & POE SURGE TANK EXCAVATIONS

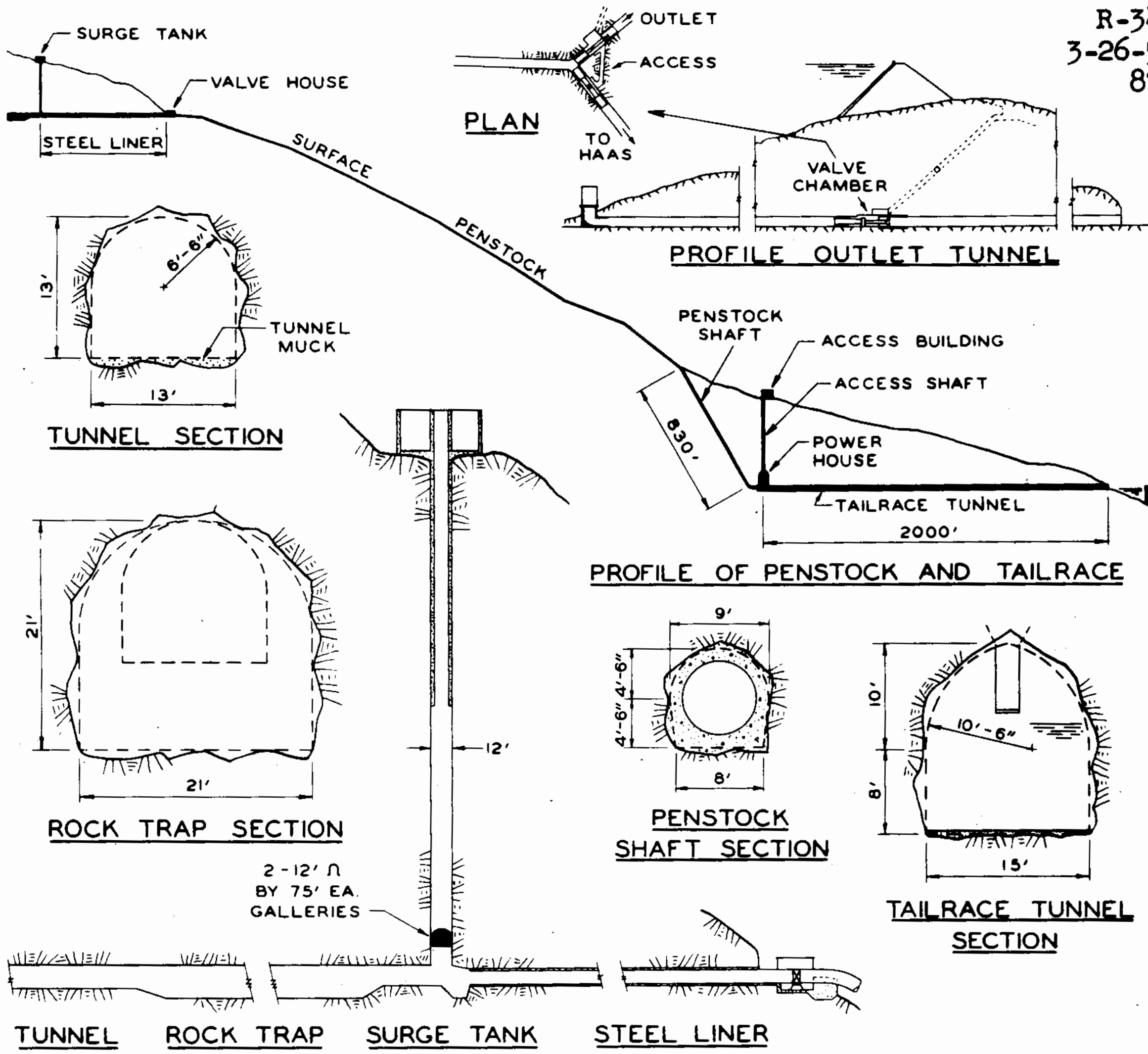
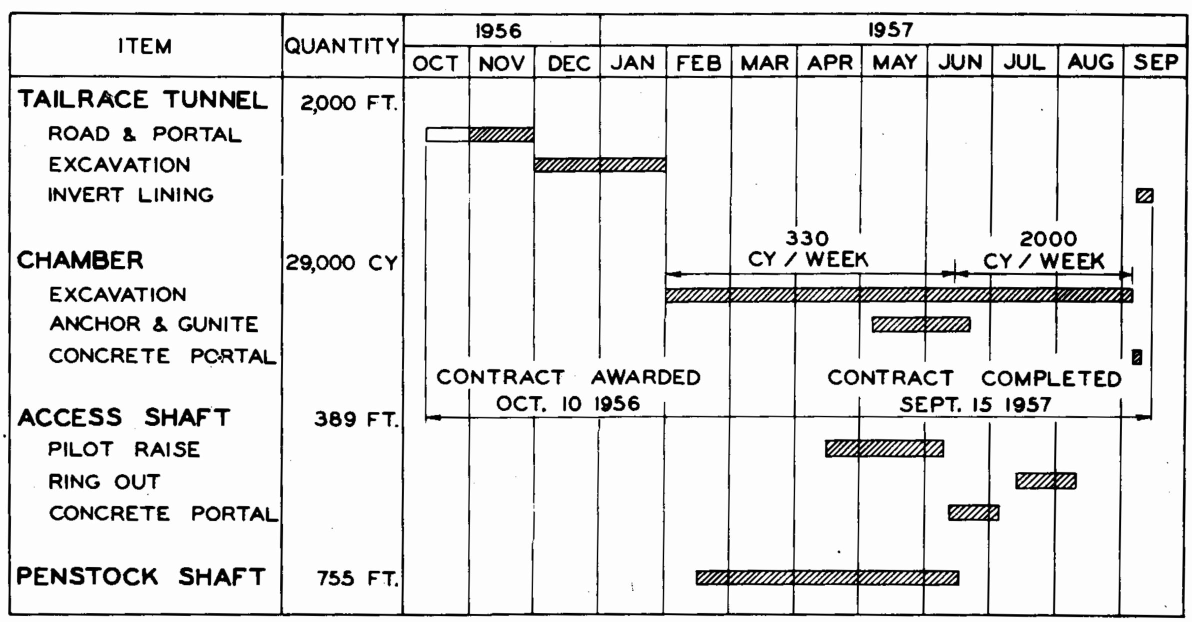
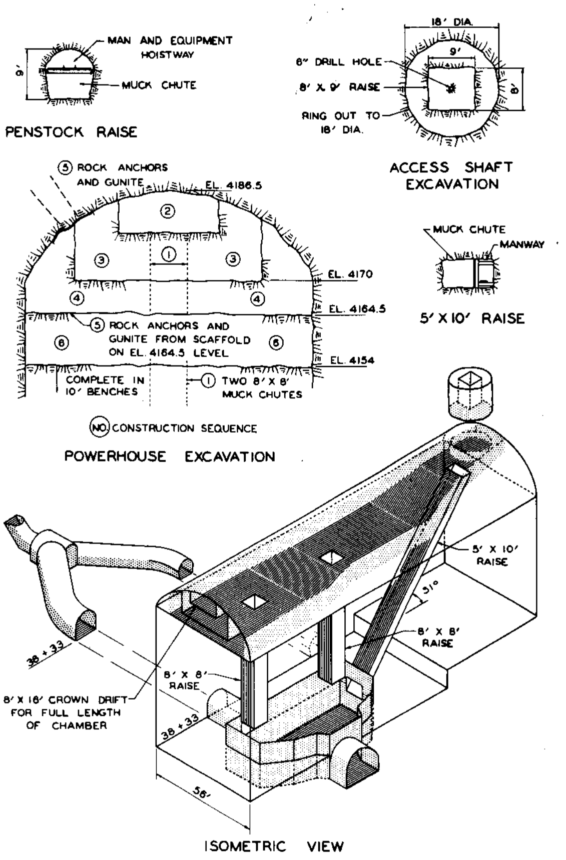


FIG. 4 HAAS PROJECT EXCAVATIONS



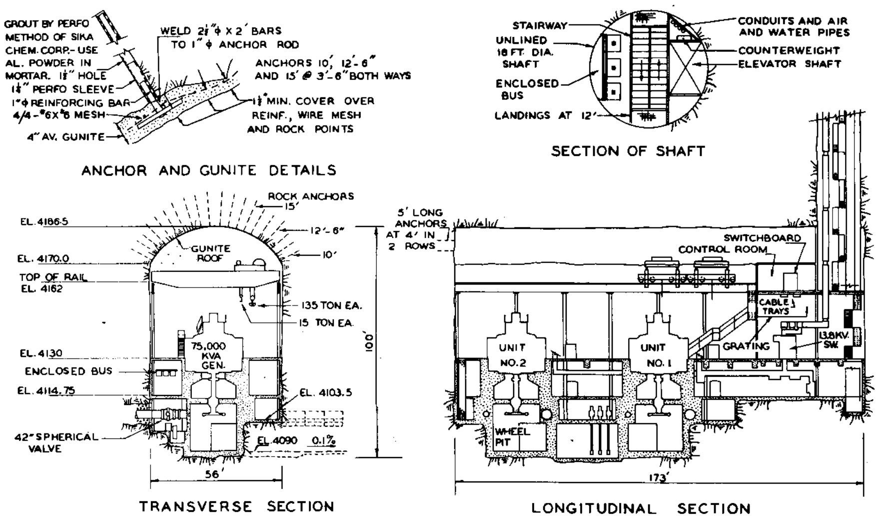
NOTE: WORK WEEK IS 3 SHIFTS PER DAY AND 6 DAY WEEK.

FIG. 5 HAAS POWER PLANT- EXCAVATION PROGRESS



HAAS POWERHOUSE EXCAVATION

Fig. 6



SECTIONS AND ROOF DETAILS — HAAS POWERHOUSE

Fig. 7

Fig. 8

Fig 9

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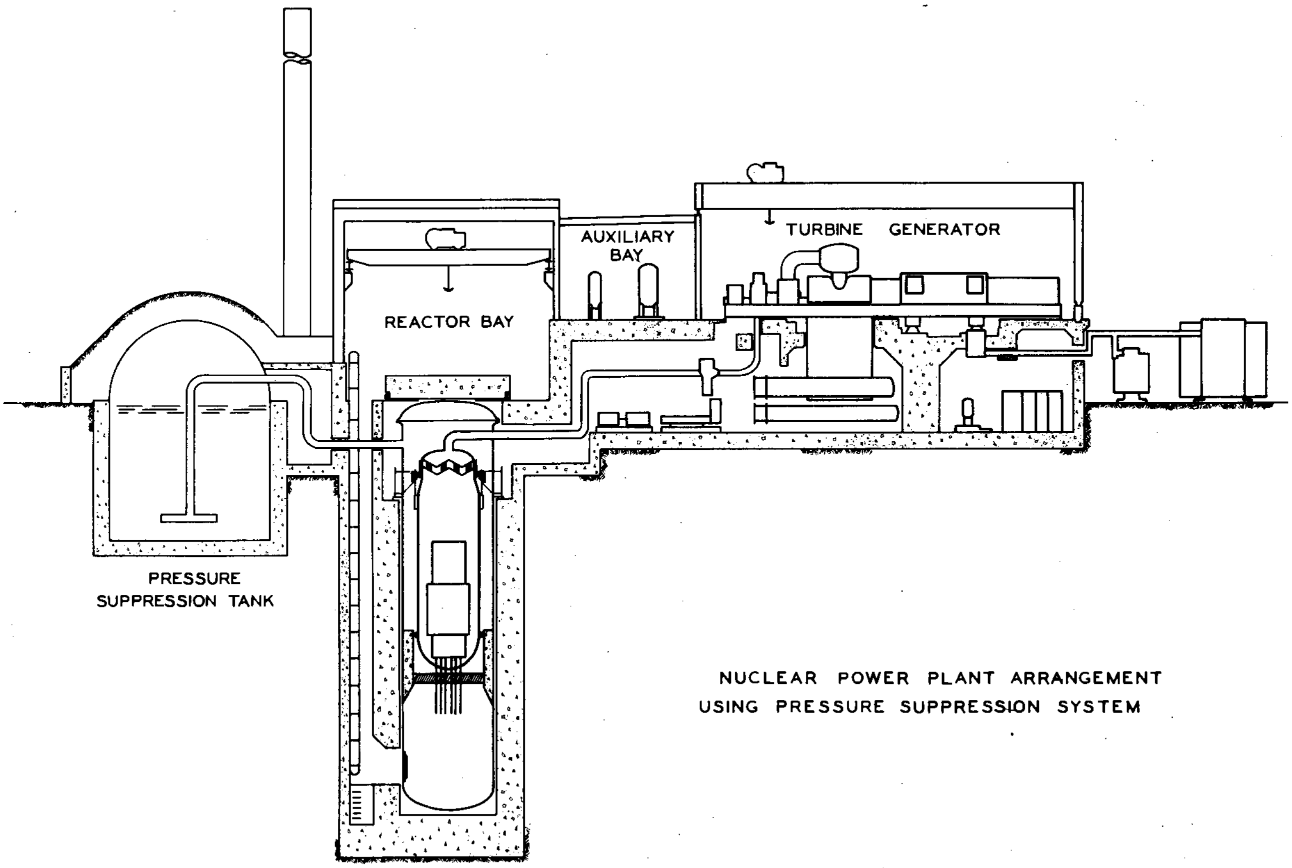

NUCLEAR POWER PLANT ARRANGEMENT
USING PRESSURE SUPPRESSION SYSTEM

Fig. 10

SWEDEN UNDERGROUND

(INTRODUCTION TO THE SWEDISH STATE POWER BOARD FILM "RIVER UNDERGROUND")

Lars de Jounge
Sandvik Steel, Inc.

1. There are many natural reasons why Sweden is quite advanced in having underground installations:

   a. The location close to the Iron Curtain.

   b. With a neutral policy, we do not want to depend on anyone.

   c. All oil, gas and coal has to be imported.

2. Mining has always been a big industry with a large number of experienced people available. When World War II started the most important defense manufacturing was put underground, e.g., munition and explosive manufacturing, complete airplane manufacturing.

3. Since the war all efforts have been made to secure and protect our defense machinery. I would like to say that Sweden then was in the same position in which the U.S.A. is now. If there was going to be another war it could be fought in our country. The following vital installations have been put underground:

   a. Docks for destroyers and submarines.

   b. Hangars for interceptors and light attack planes.

4. For the protection of civilians as well as military personnel many air raid shelters were built. The government insists that practically every large housing project have some kind of an air raid shelter. There have been 20,000 standard subterranean shelters built that also give fairly substantial protection against atomic bombs. Some 2,000 standard shelters are being provided for an additional 1.3 million people each year.

One of the biggest shelters is used in peacetime as a garage. Essentially, the layout consists of a 1,600-ft tunnel, 42 ft wide and 33 ft high. In the event of an emergency this shelter can be used by 20,000 people.

5. Because all oil, gas and coal has to be imported, many big underground storages have been built. Different methods have been developed to keep gas and oil stored directly in rock chambers without loss or contamination.

6. Our electricity is mostly hydropower and these power plants have been put underground or at least been made as secure and safe as possible.

Figures 1 and 2 show site plans of the construction of the tailrace tunnel for the Stornorforrs plant.

The three, ultimately four, generators have a capacity of 150,000 KW each, and are the largest units constructed thus far.

Figure 3 shows the drillers' cage for driving raises.

In Sweden we take pride in pointing out what we feel is the biggest tunnel in the world. But I must be honest with you and say that the Stornorforrs tunnel no longer has the biggest area. The machine halls of the complete underground 700,000-KW steampower plant known as Stenungsund are bigger. (Fig. 4). Each of the six machine halls are made as a tunnel with a length of 409 ft, width 80 ft and height 109 ft. The area is 4,400 ft^2 which exceeds Stornorforrs by 200 ft^2 (Fig. 5).

7. You might now think that costwise these underground installations will be much higher than so-called "normal costs." A few examples might be of interest. The cost of the development at Stornorforrs is $112 per installed KW, one of the lowest figures recorded in Sweden for a postwar development.

The drilling, blasting and mucking of the tailrace was $3.15 per cubic yard for the top heading, and including benches $2.45. Administrative costs are not included.

Civilian installations can be put underground economically. The Swedish wines and spirits monopoly just have had the first winetanker unloading directly into their new 10.5 million gallon underground cisterns.

FIGURES

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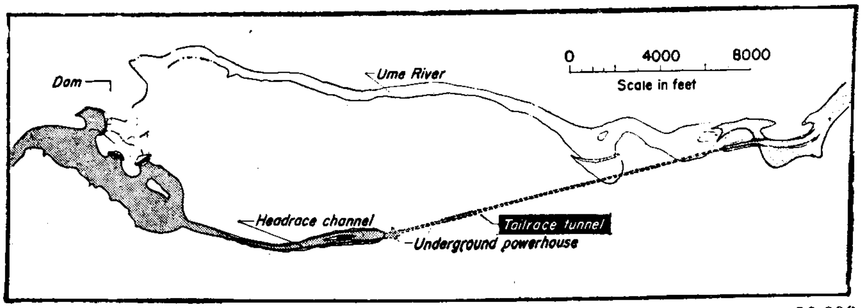
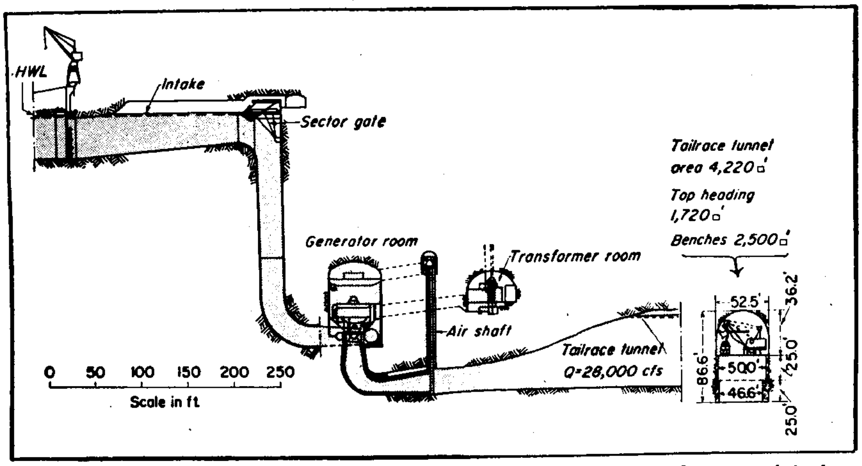


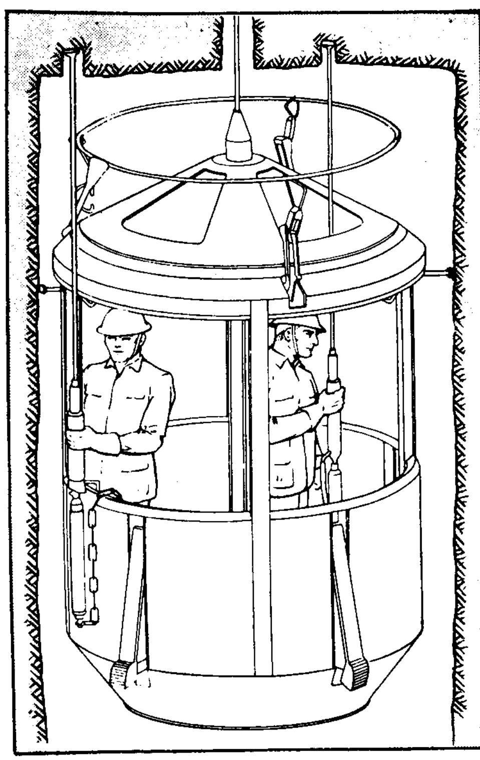
STORNORRFORS PROJECT features 2.5 mile tailrace tunnel designed to convey 28,000 cfs of water from four 178,000 hp Francis turbines, which are among world's largest.

Fig. 1



UNDERGROUND ROCK EXCAVATION totals 2.2 million cu yd, most of it from world's largest tunnel, the 52.5 x 87 ft tailrace which was driven by top heading and benching in two stages.

Fig. 2



DRILLERS' CAGE for driving penstock raises to 8.5 ft dia. Raises were enlarged to 26 ft dia from top down.

Fig. 3

Fig. 4

Fig. 5

UNDERGROUND PHENOMENOLOGY

H. L. Brode, Chairman
(The RAND Corporation)

I. SUMMARY AND INTRODUCTION

In conjunction with the Second Symposium on Protective Construction, held at The RAND Corporation (24-27 March 1959), a relatively small group of technical people met to explore the present status and the probable future of research into the underground phenomena associated with air or surface burst nuclear explosions. Emphasis in the main symposium was on construction techniques and practical problems, while the small working group was principally concerned with the physical phenomena related to the survival or failure of deep underground structures.

This effort was prompted by the need for a review of present knowledge of the basic phenomena, and more importantly by the need for suggestions as to how the physical understanding of these phenomena could be improved. The range of pertinent phenomena, extending from the earliest phases of cratering, through the propagation of various waves and including the interaction of these waves with underground structures, is very broad. For this reason, and also to facilitate productive discussion, the group was arbitrarily divided into three sessions, meeting separately at least part of the time, to concentrate on specific portions of the phenomena. The groups, with their chairmen, are listed below:

CRATERING GROUP

Dr. Curtis W. Lampson, Chairman--Ballistic Research Laboratories

Mr. Donald Anderson--Armour Research Foundation

Dr. Robert Bjork--The RAND Corporation

Dr. Harold L. Brode--The RAND Corporation

Dr. Nancy Brooks--The RAND Corporation

Commander W. J. Christensen--Bureau of Yards and Docks, USN

Dr. J. E. Hill--The RAND Corporation

Dr. M. Kornhauser--General Electric

Dr. John G. Lewis--Armed Forces Special Weapons Project

Dr. J. L. Merritt--University of Illinois

Dr. B. F. Murphey--Sandia Corporation

Dr. Gene Pelsor--University of California Radiation Laboratory

Mr. Beauregard Perkins--Ballistic Research Laboratories

Dr. John S. Rinehart--Colorado School of Mines

Dr. Donald Sachs--Stanford Research Institute

Dr. Ted Schiffman--Armour Research Foundation

Dr. Richard Skalak--Columbia University

Dr. Fred Smith--Colorado School of Mines Research Foundation

WAVE PROPAGATION GROUP

Professor Daniel C. Drucker, Chairman--Brown University

Dr. Douglas Anderson--The RAND Corporation

Dr. Millard F. Barton--Space Technology Laboratories

Professor Hans Bleich--Columbia University

Dr. Chi-Chang Chao--Stanford University

Professor J. W. Craggs--Brown University

Dr. Frank L. DiMaggio--Columbia University

Mr. Wilbur Duvall--U.S. Bureau of Mines

Dr. Samuel Genensky--The RAND Corporation

Dr. B. F. Howell, Jr.--Pennsylvania State University

Dr. Carl Kisslinger--St. Louis University

Mr. Robert Loofbourow--Mining Engineer

Dr. E. L. McDowell--Armour Research Foundation

Professor Julius Miklowitz--California Institute of Technology

Professor Nathan Newmark--University of Illinois

Dr. Blaine R. Parkin--The RAND Corporation

Professor John Rinehart--Colorado School of Mines

Dr. Fred Sauer--Stanford Research Institute

Professor Werner E. Schmid--Princeton University

UNDERGROUND STRUCTURES GROUP

Mr. Paul Weidlinger, Chairman--Weidlinger Associates

Dr. Lawrence Adler--Michigan College of Mining and Technology

Dr. Melvin L. Baron--Weidlinger Associates

Professor Stefan Boshkov--Columbia University

Mr. William Brown--The RAND Corporation

Mr. Albert Cahn--The RAND Corporation

Professor Freeman Gilbert--University of California, Los Angeles

Professor Niles Grosvenor--Colorado School of Mines

Dr. John D. Haltiwanger--University of Illinois

Mr. William R. Judd--Geotechnical Consultant

Mr. John Lynch--Office of Civil and Defense Mobilization

Mr. Hubert Moshin--The RAND Corporation

Mr. Thomas Morrison--American Machine and Foundry Company

Mr. David Singer--Armour Research Foundation

Dr. R. B. Vaile, Jr.--Stanford Research Institute

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Mr. Eric Wang--Air Force Special Weapons Center

Dr. Merit P. White--University of Massachusetts

The body of this report consists of the separate conclusions of each of the three sessions as prepared by each chairman. Although the summaries and conclusions of each of the sessions were arrived at separately, and on most points represent the consensus of that session, the present form of this report was reviewed by the entire working group.

II. COMMITTEE REPORT ON CRATERING AND RUPTURE

1. The Committee is tremendously impressed with the hydrodynamic model crater calculations of Drs. Brode, Bjork and Brooks and urges that: (1) the calculations be carried to a level of pressure at least equal to the detonation pressure of an HE charge and as far beyond as seems reasonable to the group; (2) a similar calculation be carried out for the case of a nuclear charge at two or more depths below the surface, of which one, at least, is deep enough so that surface effects are negligible; and (3) the same calculations be repeated for an HE charge to assist in the scaling process between HE and nuclear charges underground.

2. The Committee urges that additional calculation and analytical work be instituted to investigate the region between the crater and the limit of permanent deformation of the material. It is essential that such analytical work be coordinated with experiments designed to check the validity of the calculations. The Committee is cognizant of the difficulties of the task but believes that as many avenues of approach and <u>skillful</u> uses of approximations as can be found should be applied to this difficult problem.

3. The Committee believes that developments of the exploding wire technique for the production of explosive pulses and controlled media, such as plaster of paris, should be utilized for the laboratory investigation of the cratering and rupture zone formation. These results should be compared with micro-explosive charge experiments in the same medium.

4. The Committee believes that methods of investigation of the stress-strain curves of materials such as rock in situ at appreciable depths can

and should be developed in order to provide useful input data for the analytical and computational approaches to the problems.

5. The Committee urges that pressure, and material motion data directly underneath a surface-detonated charge be acquired. There appears to be a great gap in our information concerning this region. This gap, which has occurred because of the difficulty of making such measurements, must be filled as soon as possible.

6. The Committee suggests that developed geophysical prospecting methods such as refraction shooting and resistance survey methods, in addition to the sand column technique, be exploited in order to delimit the zone of rupture and permanent deformation as accurately as possible.

7. The Committee urges that refraction information and whatever other underground measurements seem practical be incorporated in the forthcoming Plowshare operation.

8. The Committee believes that an excavation of the Jangle "S" crater should be conducted (if it still exists) with the objective of determining the rupture zone and true crater size.

9. Addressing ourselves to the problem of desirable courses of action in the three postulated situations, viz, (1) no future nuclear tests, (2) deep underground tests only, and (3) possible future nuclear surface tests, we arrived at the following conclusions:

a. No future nuclear bursts:

A series of tests, similar to the U.E.T. series in Utah, using moderate-sized explosive charges of special shapes, should be conducted in a rock site, with the express objective of measuring the effects directly beneath a charge on the surface and at various heights close to the surface. This test series should utilize the results of the RAND calculations and whatever other analytical approximations are developed as a mutual check and guide for the instrumentation.

b. Deep underground shots only:

We believe that drilling and refraction techniques should be explored with the objective of delimiting the zone of rupture and permanent deformation, if possible, in order to compare with the planned RAND calculations of the radii of these zones. Measurements of acceleration, pressures, and pulse arrival times are assumed to be a part of such experiments in any case.

c. Nuclear surface burst:

The same suggestions as for no testing or limited testing, with emphasis on measurements directly under the charge. Crater sectioning the sand column techniques should be applied in addition. A great need was expressed for the development of a small, self-recording pressure gauge to be buried in the sand column and later recovered when the sand column is excavated.

ADDENDUM:

Considerable interest was expressed on the applicability of high-speed-impact cratering information to the nuclear cratering problem. The advantage of this method of crater formation, if it proves to be applicable, is that the energy density at the impact point can be varied over wide ranges in contrast to the use of HE for this purpose. RAND calculations should shed light on the problem of applicability.

III. COMMITTEE REPORT ON WAVE PROPAGATION

The Underground Wave Propagation Group attempted to establish the information which is presently available on the free field problem* and to determine the direction in which present and future research should go. As the discussion progressed it became apparent that definitive conclusions on the problems of immediate interest could not be formulated, because much of the necessary basic information has never been determined.

We started, therefore, with an analysis of the types of problems to be studied. Most attention was devoted to two special cases, the near miss and the direct hit, Fig. 1.

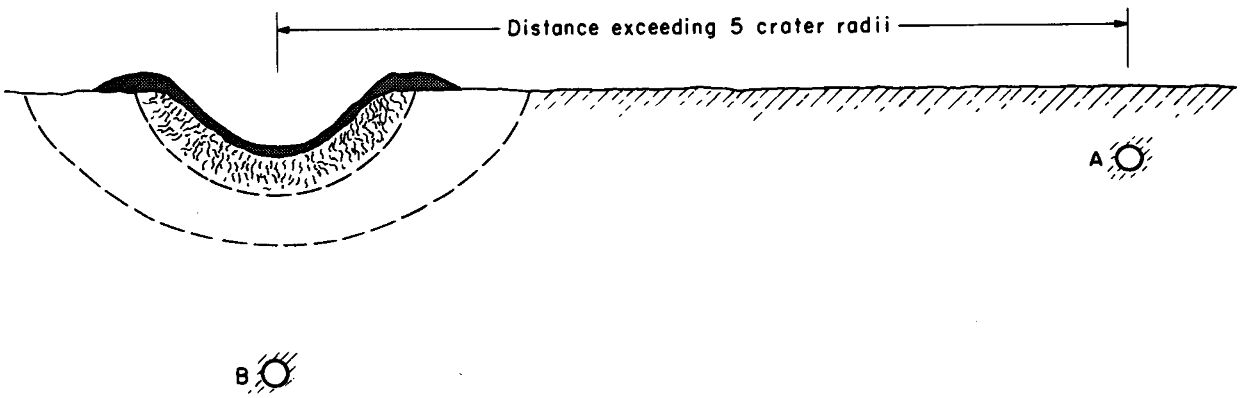


Fig. 1—Schematic diagram showing relative positions of underground shelters and surface burst crater for a near miss at A or a direct hit at B

NEAR MISS PROBLEM:

In Fig. 1 the point A would correspond to the relative position of the target structure with respect to the crater from a surface burst. In this case the majority opinion was that the important loads on the

*The term free field wave propagation is defined to include the study of underground wave motions in regions of the medium which are not influenced by man made structures or, in a medium which contains no such structures.

structure would be induced from the expanding air blast. Others felt that important loads on the structure may also be transmitted through the earth directly from the crater zone. Rayleigh waves in particular may cause trouble. It was concluded that both possibilities should be examined.

A number of competent investigators are studying linearly elastic wave propagation in connection with this problem. Work is being carried out for the isotropic homogeneous elastic half space, and some plan to include the effects of variation of properties with depth and also to solve layered systems.

The wave propagation group recognizes the need to consider inelastic bodies as well, but feels that the elastic solutions will be an extremely useful guide for design and are a necessary first step in the problem of the near miss. A second step would be to assume a visco-elastic or a perfectly plastic material, but the mathematical difficulties are overwhelming.

DIRECT HIT PROBLEM:

The point B of Fig. 1 shows the position of the underground target with respect to the crater after a direct hit. The wave propagation group also considered the case in which the shelter was located off the vertical axis of symmetry. They could not find any simple problems which might aid the designer but looked forward to the axially symmetric elastic wave solution considered above.

In the case where the target is centered directly under the crater it appears that there are some relatively simple idealized problems which one may study in order to gain a physical feeling for the effects of real materials and the inhomogeneities which are present in the earth's crust.

The actual problem has pressures applied to a cavity at the surface of a half space (Fig. 2). The replacement, which is far simpler, is a cavity in an infinite body (Fig. 3). Interaction between the waves and the actual free surface are thus ignored.

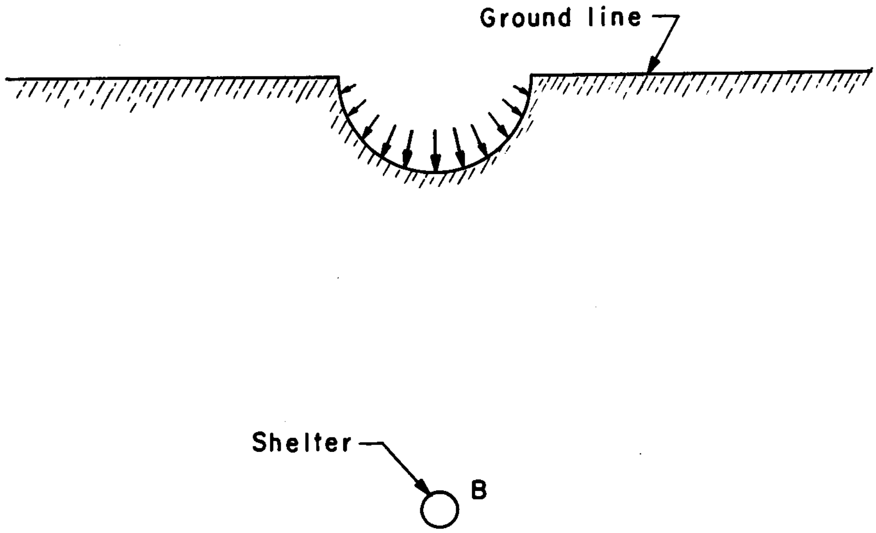


Fig. 2—Pressures acting on a spherical cavity in a half space

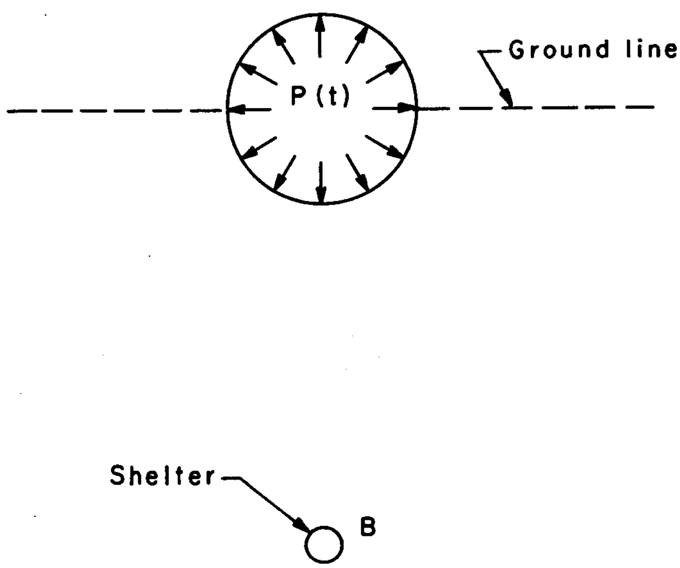


Fig. 3—Problem reduced to one of spherical symmetry by omitting the presence of the ground surface

When results from the cratering study become available they should be used in the formulation of the boundary conditions on the spherical cavity. Also, as shown in Fig. 4, the center of the cavity may be displaced upward from the ground surface position when interpreting the meaning of the solution, if the results from the cratering study make this step seem desirable.

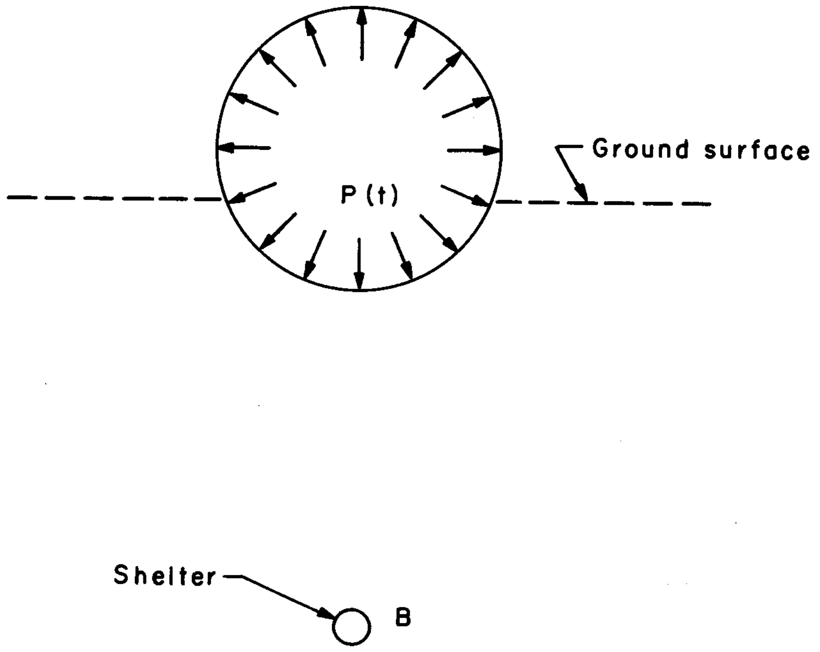
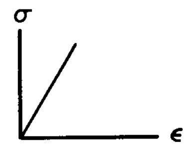


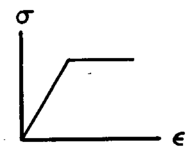
Fig. 4 — Center of cavity displaced from ground surface

The range of problems to be considered should include inhomogeneous as well as homogeneous media. Examples are the propagation of spherical waves through a medium whose properties vary continuously with radial distance, a medium composed of homogeneous spherical layers, and perhaps also a medium containing inhomogeneous layers. Various idealized materials should be assumed which under appropriate conditions simulate the salient features of the behavior of actual materials in the earth. A list of such materials together with schematic stress-strain diagrams follows:

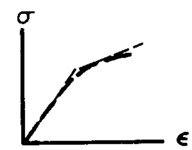
1. Linearly elastic



2. Elastic perfectly plastic

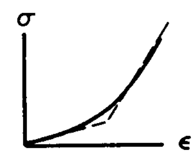


3. Elastic work hardening

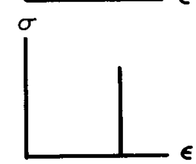


4. Linear visco-elastic

5. Locking material



6. Perfectly inelastic locking material



7. Visco-plastic

Although the plane-wave problem is not much simpler than the spherical and is farther from physical reality, the group felt that much of value could be learned from its solution as well. An intuitive understanding of what is likely to happen to a complex wave requires as much of this background information as possible.

The group turned its attention next to the question of what experiments should prove valuable and how they might be performed. Possible experimental work was divided into three groups:

1. Laboratory determination of stress-strain relations from intermediate to very high rates of loading and with little

to very high lateral restraint. The suggestion was made that explosive charges be used and that both the plane strain case (thin disk) and the long bar be employed. Tests in situ are then required to establish the correspondence between laboratory and field.

2. Comparison of experimental results with the theoretical predictions which are now known and those which will become available. Stress-strain relations established by (1) are to be employed for simple one-dimensional problems of plane wave propagation and spherical symmetry discussed previously.

3. Predictive experiments should be attempted to provide direct design information. This is the last step in the sequence of experiments and both in situ and laboratory experiments should be considered. The group as a whole felt that the possibility of modeling on a laboratory scale was remote but should not be discouraged. They also pointed out that valuable basic design information could be obtained from specific tests in situ which were not necessarily model tests, nor full scale.

A considerable discussion was held on the matter of instrumented field tests designed for some immediate purpose unconnected with the free field problem and for which little time could be spent on planning for this purpose. It was felt very strongly that money should not be spent <u>in this manner</u>. Well planned and instrumented free field tests are essential and should be carried on simultaneously and with equal priority.

Although much time was spent on the question of instrumentation, and the enormous difficulties of measurement in the free field were the subject of repeated comment, it was felt premature to suggest appropriate instrumentation at this time. Stress measurements in the free field are especially difficult and seem to be impossible at present.

Mr. Wilbur Duvall brought to the attention of the session some experimental results, obtained by means of high explosive charges, on the propagation of waves in rock. These measurements were made on the materials in situ and most observations were carried out at stress levels which one might have expected to be within the elastic regime for the rock. However, it was found that stress pulses decayed with distance nearly as $1/r^2$. Had the medium behaved elastically the attenuation with distance would have been proportional to $1/r$, so that these results show a marked deviation from linear elasticity. Moreover in one sandstone layer it was found that the wave front steepened to form a shock.

Several times during the two-day meeting, attention was turned to the influence of the expanding air pressure pulse applied to the surface, on the stresses induced by a direct hit. The feeling of the Chairman was that an order of magnitude calculation demonstrated that significant stresses could not arise from these pressures. Two figures were compared. One, Fig. 5, showed the impulse induced by the air pressure, which Dr. Brode* has computed to be in the neighborhood of 500 psi-seconds. The other was obtained from extrapolation of results for water as given in the monograph edited by Glasstone, The Effects of Nuclear Weapons, 1956, Government Printing Office, page 217 Fig. 5.48. At a distance of 0.2 mile from

*H. Brode, "Space Plots of Pressure, Density and Particle Velocity for the Blast Wave From a Point Source in Air," The RAND Corporation, Research Memorandum RM-1913-AEC, June 3, 1957.

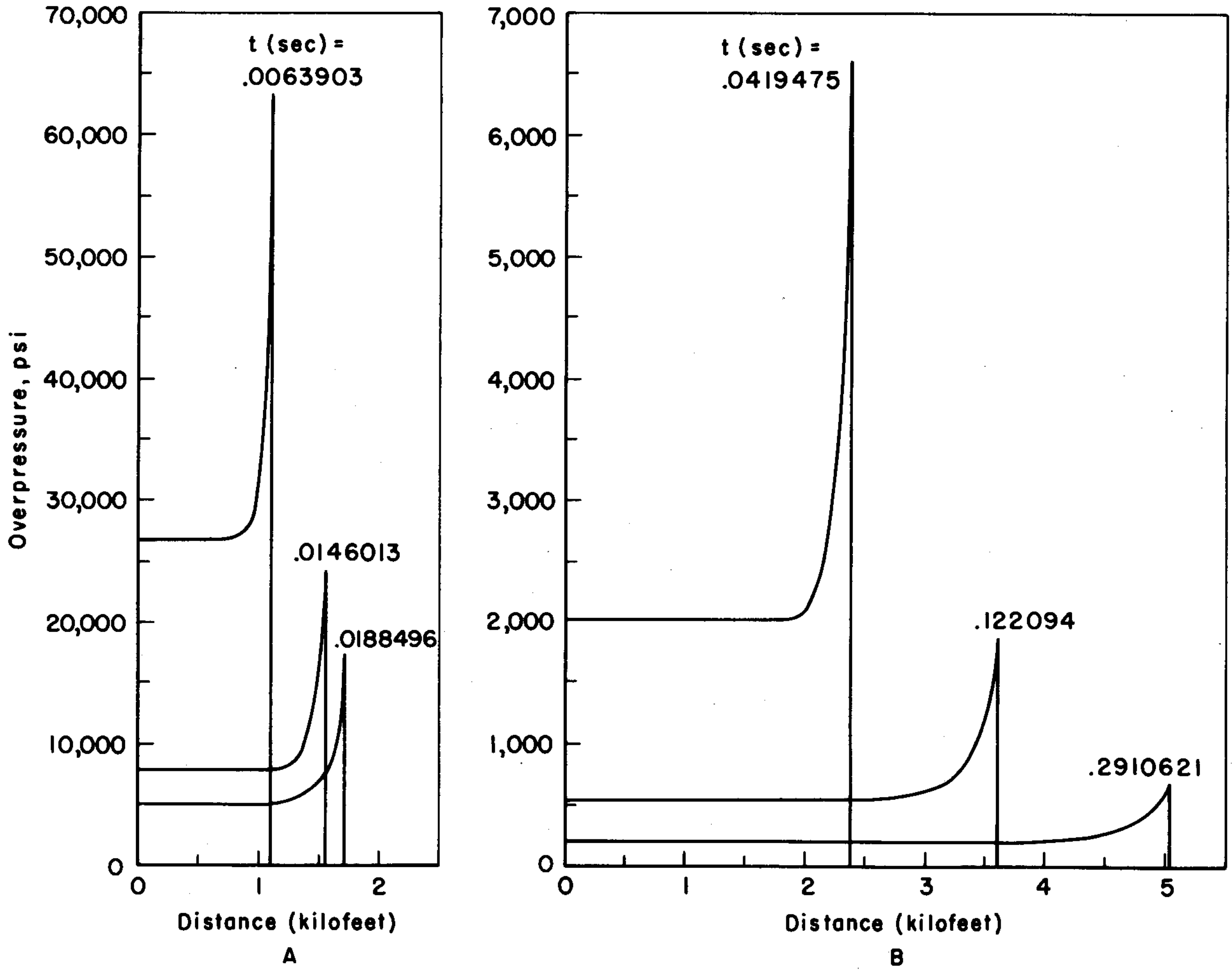


Fig. 5—Space plots for blast wave from a 20 megaton surface burst at various times after detonation

(A) Very intense pressures produced at very early times after detonation

(B) Pressures in blast wave at later times surface zero a one kiloton explosion in deep water is said to produce 30 psi-seconds impulse and over 2000 psi. Using the scaling factor $W^{1/3} = 27$ for a 20 megaton burst, 810 psi-seconds at 5.4 miles is indicated. The impulse in water varies approximately as 1/r so that the impulse at 0.2 mile should be at least 20,000 psi-seconds. A comparison of the 20,000 with the 500 for the air pressure indicated to the Chairman a strong probability that the air pressure could be ignored. Furthermore, the pressure which scales to 2000 psi at 5.4 miles would be over 50,000 psi at 0.2 mile as compared with mean air pressures of the order of 1000 psi on the surface and more than one psi static stress for each foot of depth. It did appear correct to assume that the impulse transmitted to the water in a deep burst would be of the same order of magnitude as the impulse transmitted to the soil or rock in a surface or sub-surface burst. Attenuation in soil or rock would be likely to be much greater than for water but the chairman felt that stresses of 10,000 psi at a depth of 3000 feet were a strong possibility.

The group, however, felt that this question was a very difficult one which required much additional study, and it suggested that the comparison be made for an elastic solution in an effort to obtain an answer.

It is worth noting, in closing, that, although there are so many unsolved problems of great difficulty and so little is known about the physical properties of rock and of soil, the group felt optimistic about the successful accumulation of sufficient information for a rational design of underground structures. All were especially pleased by the report of Mr. Robert Loofbourow on available geological formations. Once it becomes clear that a particular geological configuration is desirable for the given application, it appears to be true that the site can be found in nature.

IV. COMMITTEE REPORT ON STRUCTURES RESPONSE

In this session four informal, but previously prepared papers, were given. Discussions were then directed to the topics of the papers. A portion of the remaining sessions was concerned with the discussion of a series of questions which were previously prepared by the chairman.

The first two papers discussed the responses of a cavity in an elastic space which is subjected to an incident elastic wave.

The first paper, by Dr. Baron, deals with a cylindrical cavity subjected to a constant-amplitude, incident stress wave carrying bi-axial stresses. The problem is solved by superposition of the stress field without a cavity and the stress field required to produce a traction-free cavity boundary. Computations are carried out by using the mode approach. Numerical data for the n = 0 mode are given for an incident wave of constant amplitude. The general method for higher modes is also given.

The second paper, presented by Professor Freeman Gilbert, considers an approximate solution of elastodynamic diffraction and scattering problems. This method is an extension of the geometrical theory of diffraction by Keller, 1958, and an extension of the first motion hypothesis (Knopoff and Gilbert, 1959). It depends on a modification of the usual laws of geometrical optics. From ray theory, modified to account for diffraction, the first term in the asymptotic series representation of the total solution to a problem can be determined even if the boundary shape of the scatterer is not a member of a separable coordinate system.

The solution gives sufficiently accurate results for high frequency components near the wave front. To obtain solutions of the field behind the front additional terms of the series are required, and the method may lose some of its advantageous features. On the other hand, a special advantage of the method is that it is suitable for the solution of diffraction problems around smooth convex cavities of shapes which are not coordinate surfaces of the system.

The discussion of these papers centered around two main issues. First: can the proposed computations lead to useable numerical data in the near future? Both authors were able to assure the group that numerical calculations can be executed without extraordinary technical difficulties in relatively short time.

The second question was whether the assumption of an elastic medium as representing rock is tenable and realistic.

It was agreed that the elastic assumption will lead to a first approximation and that this problem must definitely be solved before other media can be considered. While some doubt was expressed regarding the feasibility of finding solutions for nonelastic cases, Gilbert suggested that his approach may be adopted with modification to visco-elastic media.

The third paper, by Mr. Singer, gave a summary of a study of the vulnerability of deep underground shelters in rock which had been prepared for the Office of Civil and Defense Mobilization. Free field data and crater dimensions are obtained by assuming spherical wave propagation, due to the application of a pressure pulse to the surface of a spherical cavity in an infinite homogeneous, isotropic elastic medium, (Selberg 1952). It is assumed that if the center of the sphere is located at an appropriate distance above the free boundary of an elastic half space, the spherical waves in an infinite medium might approximate the actual free field phenomena in the elastic half space. The effect of the resulting stress waves on the cavity itself is obtained from the static stress concentration factors obtained by loading of an elastic plate containing a circular hole. Dynamic photo-elastic methods are discussed, but the speaker stated that while these experiments produce fringes similar to those of static tests, no estimates were currently available regarding the intensity of the principal stresses from the dynamic loading due to an incident plane wave.

Numerical computations have indicated that openings of about 35 feet diameter, placed at a depth of 800 feet below ground zero in strong rock, may be damaged by a surface burst in the kiloton range.

During the discussions, the consistency and the appropriateness of the methods were questioned. The speaker stated that these computations are at best approximate, yielding only order of magnitude information. It was brought out, however, that a similar classified investigation conducted a number of years ago by American Machine and Foundry led to results which were comparable to the data presented.

The next paper, by Dr. Vaile, concerned itself with the problem of repeated shots. He stated that most structures will be of such nature that pressure ranges, under which they suffer no damage or total damage, differ at most by a factor of two. He further pointed out that the probability of experiencing loads within a factor of two of the collapse load, considering CEP's and overpressure decay rates, is small for multiple shots. A structure which will survive a single shot is very likely not to be affected by a number of equal or sufficiently lesser intensity load applications. He concluded, therefore, that design for single shot survival was a reasonable procedure.

The discussion brought out that these computations are directly confirmed by the inspection of the usual formulas which give the probability of kill assuming a standard distribution of the individual shots, but that the conclusions are realistic only if none of the shots is of such intensity that it will produce residual stresses, that is, no yielding will occur. However if yielding should occur the stresses produced by further load applications will be additive to previous residual stresses. This is always the case in statically indeterminate structures, and since the cavity itself must be considered as highly indeterminate, it may be advisable to set the vulnerability of such tunnels at a level which will insure that elastic stresses only will be produced. On subsequent discussions by the three groups it was expressed that it may become difficult under these assumptions to design underground tunnels which may be located directly under ground zero because of the estimated high intensity of the incoming stress waves.

Further discussion ensued on the problem of failure mechanisms and failure initiation. It was the consensus of the group that at the present time there is no sufficient theoretical or practical background to conjecture regarding the most realistic strength theory to be applied. There was some feeling that direct tension failures may be of significant importance. It was also proposed that a realistic lower bound may be obtained by neglecting the tensile strength of rock. This assumption is especially justified if the presence of closely spaced and scattered tensile cracks exist initially in the rock. During further discussion within the three groups it was proposed that it may be proper to design on the assumption that net tension around the opening would be zero. That is, the superposition of gravity, tectonic and dynamic stresses due to the incident wave should, at most, produce zero tension around the cavity.

The remaining parts of the session were devoted to a series of specific questions. The first set of questions were concerned with the effect of discontinuing nuclear tests. It was the opinion of the group that if nuclear tests are not to be conducted in the future the resulting loss of information can at least partially be replaced by intensified theoretical work, laboratory tests, and high explosive tests. It was also agreed that if nuclear tests are to be conducted, the selection of a more suitable test site more pertinent to the problem at hand, together with the explosion of multimegaton weapons should lead to the most desirable type of experiments. On the other hand, it was felt that small-yield kiloton tests or high explosive tests must be interpreted with a great deal of caution and should not be extrapolated to high yields by the usual and rather crude scaling methods. The group also felt that the greater difficulties which are usually encountered in the instrumentation of such tests and the impossibility of conducting repeated explosions make the carefully controlled and measured laboratory experiments seem quite attractive, and sufficient attention should be paid to these. The second group of questions considered the type of theoretical investigation which should be practiced. Intensified investigations should be conducted both in elastic and nonelastic media to determine: stress and velocity fields in the neighborhood of the cavity; failure criteria and failure mechanisms; effects of an elastic boundary in the cavity (i.e. linings); and shock effects on the contents of tunnels.

It was suggested that these investigations, if possible, should be carried out numerically for an entire range of assumed peak pressures, and that this theoretical work should ultimately lead to a simplified engineering method, or at least to the extensive tabulation of data suitable for design purposes.

The group also felt that while no specific consideration was given to phenomena occurring in soil these should not be neglected, and it has recommended that similar or parallel investigations should continue for this medium also. It was also agreed that in order to give maximum meaning to numerical computations it will be essential that these be accompanied by laboratory and field tests to test the reliability of the proposed theories. At the same time it was pointed out that currently a greater need exists for the development of more suitable instrumentation.

The next set of questions concerned itself with a series of design problems. The group has attempted to express its preliminary opinion or feeling at least on a number of questions which are very essential for engineering and current planning decisions, but which can only be accurately answered after all previously proposed theoretical and experimental investigations have been concluded. With these qualifications the group was able to give only some very general guidance regarding these problems.

It was felt that although theoretical investigations initially indicate that the intensity of the stress field around the boundary of a cavity in an infinite medium is independent of the size of the cavity itself, in practice it will be preferable to keep at least one dimension as small as practicable; i.e., in order to provide a required amount of floor space it will be preferable that this be provided in the form of an essentially long tunnel, or several such tunnels instead of large dome-like excavations. This approach is also specified by certain geological considerations, inasmuch as it appears likely that the type of construction suggested above will be more easily located in horizontally bedded sound rock strata.

The next question considered was the depth of these proposed tunnels or cavities. It was felt, as expected, that deep tunnels will be preferable to shallow ones, provided that gravity pressure and tectonic stresses are taken into consideration. These seem to put a definite limit on the depth at which these structures can be built. It was also felt that the effect of shock will less likely be of importance at greater depth. On the other hand, it was felt that the shock effects could be dealt with by suitable shock mountings. The trade-off between the use of shock mountings and greater depth of overburden is probably one of economy. In connection with these two problems, the group felt very strongly that each installation should be custom built, and consequently the preparation of standard plans should be avoided. Each site needs to be carefully and separately investigated, and the size and depth of each tunnel and cavity should be determined individually.

The next question was that of the spacing of tunnels. Since the only information currently available was that of static fields around openings it was suggested that, based on this evidence alone, the spacing of tunnels should be at least one diameter and preferably more, especially if the material around the tunnels is expected to be subjected to stresses which exceed yield strength.

Considerable attention was paid to the cross sectional shape of tunnels. The group essentially felt that the evidence of certain static tests seemed to indicate that the actual shape of the tunnel doesn't have a great influence on the intensity of the stresses around the boundary. On this basis it was felt that generally rounded cross sections should be preferred. During further discussion within the three groups the opinion was expressed that, due to effects of reflection and refractions from layers below the bottom of the tunnel, it may become advisable to provide a round cross section around the bottom of the cavity itself, and the use of round or elliptical tunnels with the major axis vertical seems to have found favorable reception. On the other hand, considerations of static stresses and residual tectonic stresses again indicate that individual decisions must be made in each instance.

The next question concerned itself with the usefulness of tunnel linings, either independent of the tunnel wall or directly connected to it. The group felt that at the present time it is not able to determine the advantages and disadvantages of this type of construction, and this problem needs to be investigated in detail. It was felt that since the solution of a cavity with an elastic boundary in an acoustical medium has been worked out already (Baron, 1958) an extension of this work to an elastic medium appears to be feasible.

The group was unable to give specific guidance regarding the preferred type of medium in which tunnels should be placed. There was even disagreement regarding the advantages which may be obtained in either one of the two extreme cases considered, namely that of strong rock or soft soil.

It was felt that in connection with construction of large underground cavities, specific attention should also be paid to the vulnerability of utilities and that the engineering problems connected with construction methods should also be carefully considered. There seems to be a need for the consideration of special and ingenious methods of construction and imaginative design solutions which may alleviate some of the problems that show up in the consideration of structures protected by rock.

SOME COMMENTS ON THE WAVE PROPAGATION STUDY GROUP

Samuel M. Genensky
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I deeply appreciate this opportunity to examine the proceedings of the Wave Propagation Study Group. I had hoped to participate actively in its discussions and deliberations, but an unexpected illness made this impossible. I am indebted to Dr. Douglas R. Anderson of RAND for having the content of its sessions both taped and recorded. I would like to thank Dr. Anderson and Dr. Blaine R. Parkin, also of RAND, both of whom participated in this study group, for their comments and suggestions. These gentlemen also assisted me by explaining certain drawings and blackboard sketches which could not be clearly understood from acquaintance with the audible record alone.

One of the most interesting portions of the Wave Propagation meetings involved the presentation of drawings by Mr. Robert L. Loofbourow of geological formations which might be worthy of consideration as possible sites for deep underground construction. In these drawings Mr. Loofbourow indicated various kinds of cover that might be expected to provide unusually fine protection for an underground installation against the effects of surface bursts in the megaton yield range. While I agree with my distinguished colleagues that in the light of our present knowledge, it is impossible to make a judicious choice from among these possibilities, I believe that valuable information can be obtained from laboratory tests performed upon models which simulate as closely as possible these geological configurations and others that might be of interest but were not specifically mentioned. I am fully cognizant of the fact that most geological materials are inherently non-linear, and hence extrapolation from models of such media to full scale is both a difficult and hazardous task. Nevertheless, many of Mr. Loofbourow's suggestions are not amenable to mathematical analysis at this time and hence can not be evaluated in even a qualitative manner except by experimental techniques. I do not regard such an experimental approach as being any less sound than that taken by the engineer in analyzing a problem by the methods of classical elasticity[1] when in fact a more complicated set of constitutive relations[2] may govern the materials with which he is concerned. However,

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[1]A classically elastic material is a medium governed by the equations

$$t_{ij} = c_{ijk\ell} e_{k\ell} \qquad (i, j, k, \ell = 1, 2, 3)$$

where t_{ij} are the components of stress, $e_{k\ell}$ are the components of strain and $c_{ijk\ell}$ are a set of 81 constants. It can be shown from the symmetry of the stress tensor, the symmetry of the strain tensor and energy considerations that at most 21 of these constants are distinct. Further, for an isotropic classically elastic material, only two of the $c_{ijk\ell}$ are distinct and the governing equations may be written as

$$t_{ij} = \lambda e_{kk} \delta_{ij} + 2\mu e_{ij} \qquad (i, j, k = 1, 2, 3)$$

where δ_{ij} is the Kronecker delta and λ and μ are the first and second Lamé constants.

[2]Constitutive relations are the equations which characterize a particular continuous medium. In order to solve boundary value problems involving the particular medium, it is necessary that

in both these instances it is absolutely imperative that the limitation of the results obtained be fully recognized and readily admitted.

This study group was particularly impressed with two of Mr. Loofbourow's proposals, namely (1) placing the installation in competent rock below a glacier and (2) placing it in competent rock below an abandoned mine or planned excavation. In the former case, it was pointed out that many glaciers are primarily composed of snow and hence contain a large portion of entrained air. It was felt by many in the group that this interstitial air might provide a mechanism for absorbing a considerable portion of the energy released by a nuclear burst in addition to that which would be absorbed in vaporizing the glacial snow. While this proposal appears to have merit, it may also have serious drawbacks. For example, (1) the impedance mismatch between air and glacial snow may be much smaller than that between air and hard rock such as granite, hence a greater portion of the energy released by a nuclear weapon might be expected to pass into glacial snow than into hard rock; and (2) unless the glacier is extremely thick there is reason to believe that its effectiveness as protective cover may be lost after a single explosion. In the latter case, the group agreed

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its constitutive relations, together with a continuity equation and equations of motion (and possibly an equation of state, an energy equation, electro-magnetic equations, etc.), form a complete and consistent system.

that the blast from an atomic weapon would be considerably reduced by the presence of an abandoned mine or planned excavation located between the surface and the underground facility. Although not specifically stated, their conclusion is based upon the following reasoning: Initially, a blast wave progresses through rock as a compressive wave until it encounters a surface of discontinuity, e.g., a boundary between two dissimilar rocks or a boundary between rock and air. If we exclude the first type of interface from our considerations, as not appreciably affecting the conclusions, we see that the compressive wave progresses with some attenuation due to expansion and to the anelasticity[3] of the medium, until it encounters the roof of the mine or planned excavation. At that interface it is expected (although this must be demonstrated either analytically or experimentally) that very little energy will be transmitted across the air space to the floor of the excavated region. Hence the bulk of the energy will be carried back toward the surface of the ground as a tensile wave. However, since the maximum compressive strength of most rocks is much greater than their maximum tensile strength, for very intense blast waves the rock can be expected to fail and hence spalling from the roof of the excavation is likely to occur. I join with those members of the group who felt that

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[3]An anelastic medium is one for which strain energy is not conserved.

such an excavation can be expected to provide adequate protection against repeated attacks. However, I strongly suspect that a properly planned excavation is preferable to an abandoned mine, because in the former case, control can be exercised over its location, shape and size, so as to provide the underground installation with maximum protection.

During the first session held by this group, it became apparent that two schools of thought existed concerning the analytical approach that should be taken in solving problems involving wave propagation through rocks and soils. On the one hand, there were those who felt that these media should be regarded, at least initially, as being classically elastic. They argued that if this assumption were made then powerful analytical tools were available, such as superposition and reciprocal theorems, which can be employed to solve both simple and complex problems. They further argued that such results would be valid at large distances from the crater caused by a nuclear burst, though they probably would be of little value in the neighborhood of such a crater. On the other hand, there were those who felt that soils and rocks are inherently anelastic media and hence must be represented by constitutive relations which incorporate a mechanism for dissipating strain energy. They were supported in their stand by the fact that experiments have been performed using high-energy explosives which show conclusively that the intensity of a blast wave passing through rock decreases with increasing distance r from the point of detonation as $1/r^2$ rather than as $1/r$. Further, the stresses produced in these experiments lay in what should be expected to be the elastic range, therefore proving conclusively that it is unrealistic to characterize such rocks as elastic media.

I am very much aware of the fact that one's technical training and experience influence one's views in these matters. However, even taking this limitation into consideration, or falling victim to it, I must conclude that those favoring an-elastic models have the weight of evidence on their side. It is all well and good to say we will consider a material to be elastic, because by doing so we can solve some "interesting" problems. Unfortunately, if the material with which one is dealing is not characterized by a linear relationship between the components of stress and the components of strain, all the wishing in the world won't make it so. Admittedly, it is often much more difficult to solve problems in which the material is thought to be, say, plastic or visco-elastic, but if it should prove necessary to utilize such models (and from listening to the proceedings this appears to be the case) then we should be willing to exert every effort to carry out the required analysis.

During the afternoon of the second day of this group's deliberations much time was devoted to discussing the experimental aspects of wave propagation through geological media. Since I am not an experimentalist by training or experience, I do not wish to attempt to evaluate in detail the results of those deliberations. However, it was clear from the tapes and recordings that strong differences existed as to what ought to be measured and how and where the measurements ought to be made. It seemed to me that the only point of agreement that existed among the participants concerning these issues was that no agreement could be reached at this time.

However, while listening to these lively discussions the following question came to mind: We do not know the constitutive relations that characterize rocks and soils, and hence how can we make an intelligent selection from among the mathematical models currently available? Unfortunately, the answer to this question is that we can't, except perhaps to decide between models for which strain energy is conserved and those for which it is not. It thus seems evident to me that there exists an <u>urgent</u> need for careful experimentation to determine the constitutive relationships that characterize the rocks and soils with which we are most concerned.[4] We could then introduce these relationships into the equations of motion (and any supplementary equations we may need such as an energy equation) and with the continuity equation (and possibly an equation of state) could attempt to solve the various important boundary

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[4]It should not be construed that I feel that a single set of constitutive relations exists for all rocks and soils. On the contrary, I feel that a group of rocks or soils may be represented by a single set of such relationships, but that another group of rocks or soils may obey an entirely different set of equations.

value problems. Professor Drucker describes such problems in his summary of the wave propagation meetings.

I am fully acquainted with the tremendous difficulties that would exist in trying to solve these problems even if we were to employ digital and/or analog computers. However, I feel that the value of the results obtained from such an investigation would greatly outweight the investment in time and effort that would be necessary to achieve them. Furthermore, not only would these conclusions be of enormous value to the study of deep underground construction, but they would also find important applications in theoretical mechanics, geology and both civil and mining engineering.

I do not wish to leave the reader with the impression that I disapprove of analytical studies of wave propagation through rocks and soils at this time. On the contrary, I feel that we should solve as many of the important problems as possible using the various anelastic models that are currently available. The results obtained from such investigations would give us valuable qualitative information concerning wave propagation through those materials, and once the constitutive relationships have been determined for these media, the analytical findings would be open to quantitative evaluation. Therefore, I strongly urge that both experimental and analytical programs be supported, for only in this way can we ever hope to achieve a clear understanding of the mechanics of wave propagation through geological materials.

AN INFORMAL PROGRESS REPORT ON SOME THEORETICAL INVESTIGATIONS ON THE VULNERABILITY OF DEEP UNDERGROUND OPENINGS IN ROCK

Paul Weidlinger
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The following report represents a brief summary of the results obtained on investigations of underground phenomenology by:

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INTRODUCTION

The determination of the vulnerability of deep underground openings or tunnels presents a number of difficult practical and theoretical problems. Before the engineering design can be handled a fairly detailed understanding of the physical phenomena is essential, since practical experience on the behavior of openings under nuclear blasts is not available. It is erroneous to draw conclusions from the static behavior of mines and tunnels under gravity loading, and reasoning by such analysis may be extremely misleading. Limited experimental data based on HE tests are available, but the interpretation of these results is made equally difficult without full knowledge of all phenomena resulting from blast loading.

On the other hand, theoretical work also has many obstacles because of the nature of the problem. In order to advance these investigations, numerous simplifying assumptions need to be made, with the hope that the influence of the assumptions on the actual behavior can later be included or at least estimated.

Simplifying Assumptions

The rock is assumed to be an ideally elastic, homogeneous and isotropic semi-infinite medium. Regarding this assumption the following comments can be made:

1. Some rocks can be considered elastic and most rocks can be considered homogeneous, but there are numerous materials which show a distinctly inelastic or anelastic behavior. Results obtained by elastic analysis will not apply to these latter types of materials or at best they will be approximate.

2. Sedimentary rocks are distinctly anistropic. To these, the previous comments apply.

3. Stratification need not invalidate theoretical investigations, but the influence of layering will considerably increase and complicate numerical computations. The presence of extensive rock faults has similar implications.

4. Ordinarily, it is assumed that the boundary of the semi-infinite space is plane. In mountainous areas this assumption is obviously not realistic.

5. The medium is initially not stress-free. The effect of a superimposed gravity and tectonic stress field does not present basic theoretical difficulties, but establishment of the actual intensity and distribution of tectonic stresses is questionable.

6. Assumptions need to be made also regarding the location and shape of the opening itself. Theoretical results can most easily be obtained for cylindrical and spherical openings. Most actual cross sections can be approximated in this manner. Investigations of other specific cross-sectional shapes may be very difficult. The effect of the boundary in a semi-infinite

space also increases the difficulties, but it may be shown that openings which are a few diameters below the surface can be treated as if they were located in an infinite space. Such investigations, however, will not be applicable to shallow-depth tunnels and entrances.

7. In some instances it is desirable to provide lining to tunnels. The problem of an elastic lining is susceptible to analytical treatment.

The question arises whether these simplifying assumptions will not restrict the applicability of the result, and be of very limited use only. The answer, of course, is that initial progress, in view of the urgency of the problem, needs to be made in the direction where even a limited success is at least meaningful. Seismological investigations, which are based on the assumptions listed above, seem to give answers which are at least partially verified by measurements. Theoretical work needs to be supplemented by laboratory and field measurements, and be extended in the direction of more realistic and complex assumptions. In the meantime it turns out that the "simple" elastic half-space assumption offers formidable theoretical and computational difficulties. It is clear that before other details are considered the simplest theoretical case must be fully understood.

Statement of the Problem

The surface of a semi-infinite half-space is subjected to a point source explosion which applies a time-decaying expanding pressure wave to the surface.

In the neighborhood of the source the assumption of elastic behavior cannot be maintained and consequently such investigations may yield at best an approximate <u>determination of the crater size</u>. Similarly, the effect of the transit through the plasticized material will immediately modify the character of the disturbances. It can be assumed that at some distance outside the crater the elastic assumption is again tenable.

Since elastic space is doubly refracting, both transverse and longitudinal stress waves will be produced which by interacting with the surface will give rise to complex wave patterns. The determination of these will provide information on the essential free field phenomena.

The waves generated by the source interact with the cavity itself. Determining the stress field and particle velocities in the neighborhood of the cavity constitutes the interaction phenomena of the problem. These investigations also yield significant information regarding the effects of the blast on the contents of the tunnel by supplying information regarding the acceleration and rigid body motion of the entire opening.

Depending on further assumptions regarding brittle or inelastic behavior, knowledge of the above will lead to the establishment of the failure mechanism of the opening. In the case of unlined cavities, detailed knowledge of the failure mechanism may not be essential and the establishment of failure initiation criteria may be sufficient for practical purposes.

DYNAMIC RESPONSE OF AN ELASTIC HALF SPACE TO A CONCENTRATED VERTICAL FORCE ON THE SURFACE

The problem under consideration is illustrated in Fig. 1 in which R, ϕ are spherical coordinates, h(t) is the Heaviside step function and $\tilde{u}$ is the displacement at an arbitrary point. This displacement function is obtained using existing solutions and the dynamic reciprocal theorem of elasticity.

Dynamic Reciprocal Theorem

Consider a finite elastic body whose principal modes and natural frequencies are denoted by $\tilde{\phi}_k(P)$ and ω_k, respectively, where $\tilde{\phi}_k$ is a vector and P represents an arbitrary point in the body. Let a concentrated force $\tilde{f}_1 g(t)$ act at a point P_1 of the body initially at rest. Then the displacement at point P_2, denoted by $\tilde{u}_1(P_2, t)$ is given by

$$\tilde{u}_1(P_2, t) = \sum_k q_k(t)\, \tilde{\phi}_k(P_2) \tag{1}$$

in which

$$\ddot{q}_k + \omega_k^2\, q_k = \frac{\tilde{f}_1 g(t) \cdot \tilde{\phi}_k(P_1)}{M_k} \tag{2}$$

$$M_k = \int_V \rho\, \tilde{\phi}_k \cdot \tilde{\phi}_k\, d\tau \tag{3}$$

For a body initially at rest, the solutions of Eq. (2) are

$$q_k(t) = \frac{\tilde{f}_1 \cdot \tilde{\phi}_k(P_1)}{M_k\, \omega_k} \int_0^t g(\tau)\sin\omega_k(t - \tau)d\tau \tag{4}$$

which, when substituted into Eq. (1), give

$$\tilde{u}_1(P_2, t) = \sum_k \frac{\tilde{f}_1 \cdot \tilde{\phi}_k(P_1)\, \tilde{\phi}_k(P_2)}{M_k\, \omega_k} \int_0^t g(\tau)\sin\omega_k(t - \tau)d\tau \tag{5}$$

If a force $\tilde{f}_2 g(t)$ acts at point P_2, it similarly produces a displacement $\tilde{u}_2(P_1, t)$ at point P_1 given by

$$u_2(P_1, t) = \sum_k \frac{\tilde{f}_2 \cdot \tilde{\phi}_k(P_2)\, \tilde{\phi}_k(P_1)}{M_k\, \omega_k} \int_0^t g(\tau)\sin\omega_k(t - \tau)d\tau \tag{6}$$

Comparing Eqs. (5) and (6), it is seen that

$$\tilde{f}_2 \cdot \tilde{u}_1(P_2, t) = \tilde{f}_1 \cdot \tilde{u}_2(P_1, t) \tag{7}$$

Equation (7) is a general statement of the dynamic reciprocal theorem.

Although demonstrated for a finite elastic body, Eq. (7) holds for an infinite body as well, since it is always possible for a finite time, t, to choose a finite volume out of the infinite body for which modes and natural frequencies exist.

Application of Dynamic Reciprocal Theorem to Problem Under Consideration

Using an electronic computer, Pekeris and Lifson have obtained curves for the vertical displacement, $\overline{W}_p(R, \phi, t)$, at the surface of an elastic half-space caused by a vertical concentrated force h(t) acting in the interior (see Fig. 2).

Using Eq. (7), it is seen that

$$\overline{W}\,(R, t, \phi) = u_z(R, t, \phi) \tag{8}$$

where u_z is the vertical component of $\tilde{u}$ in Fig. 1.

Using methods similar to those of Pekeris and Lifson, it is possible to obtain curves for the vertical displacement, $\overline{W}_c(R, \phi, t)$, at the surface of an elastic half-space caused by a horizontal concentrated force h(t) acting in the interior from the formal expressions for those obtained by Chao. Referring to Fig. 3 and using Eq. (7) again, the horizontal component, u_r, of $\tilde{u}$ of Fig. 1 may be obtained as

$$u_r(R, \phi, t) = \overline{W}_c(R, \phi, t) \tag{9}$$

Preliminary Results

Using the curves obtained by Pekeris it is possible to construct the wave pattern for the loading of Fig. 1 when Poisson's ratio is 1/4. This is illustrated in Fig. 4 in which r, z are cylindrical coordinates, c is the velocity of propagation of shear waves and P, S and SP denote a dilatational wave front, a shear wave front and a "head" or Mach wave front respectively.

For angles less than ϕ_c, the SP wave does not appear. It may be further determined that:

1. At the P front, the displacement has a finite amplitude less than that which would occur under a statically applied load.

2. When the SP wave arrives, its amplitude is zero.

3. The character of the S phase depends on ϕ. For $\phi \leq \phi_{cr}$, the arrival of the S wave is marked by a finite jump in vertical displacement while for $\phi > \phi_{cr}$, there is a logarithmic infinity in the displacement.

4. For $r/z > 10$ ($\phi > 84^\circ$), a marked increase in vertical displacement occurs in the vicinity of $ct/R = 1/2\sqrt{3 + \sqrt{3}} = .9194$, which corresponds to the velocity of propagation of Rayleigh surface waves. On the surface the Rayleigh wave has infinite displacements at its front. In Fig. 4, the region of these high displacements is labled R, but except on the surface, no separate wave front exists there.

In Fig. 5, vertical displacements for a few values of r/z are plotted. Note that for large values of ct/r, the displacements approach their static values.

<u>Response for Expanding Load</u>

By integration, the response due to a circular expanding load may be obtained from that due to a concentrated force. This integration will yield displacements which do not have the unrealistic infinite discontinuities which are present in the case of the concentrated force.

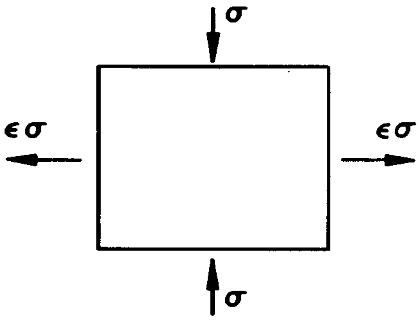
A practical procedure for obtaining both stresses and displacements when the applied pressure is an arbitrary function of space and time is presently being formulated.

INTERACTION OF PRESSURE WAVES WITH A CYLINDRICAL CAVITY IN AN INFINITE ELASTIC MEDIUM - BRIEF OUTLINE

General Statement of Problem

Once the free-field stresses are determined, the problem becomes one of determining the stress, velocity and acceleration response of the medium at and near a cylindrical cavity which is enveloped by waves with spherical wave fronts. For cavities at a large distance from the surface burst, the spherical wave may be replaced by a plane wave carrying pressure in two directions as shown in the sketch.

The pressures in the incoming wave will be taken as step functions in time. Once the free-field stresses are known, the response of the cavity to incoming pressure waves with the correct decay in time can be evaluated from the results for the step distribution by means of Duhamel integrals.

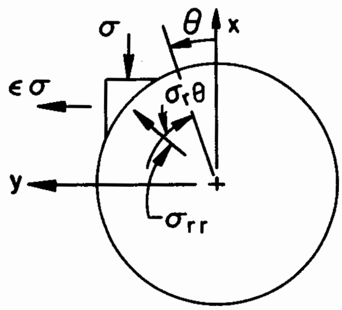


The value of ϵ is a positive or negative number depending on the loading and position of the tunnel. It is to be determined from the free field information when available.

Problem of the Unlined Cavity in Homogeneous, Isotropic Rock under Dynamic Pressure Loading

The problem is to be solved by superposition of the stress field without a cavity and the stresses due to required tractions which will make the cavity boundary surface traction-free.

A. Stress field--no cavity--due to passage of a wave with pressure components of $\sigma u(t)$ and $\epsilon\sigma u(t)$.*

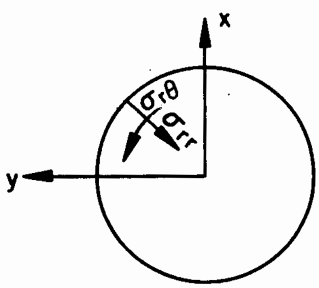


$$\sigma_{rr} = - \sigma(\cos^2\theta - \epsilon\sin^2\theta) \qquad (1)$$

$$\sigma_{\theta\theta} = - \sigma(\sin^2\theta - \epsilon\cos^2\theta) \qquad (2)$$

$$\sigma_{r\theta} = \frac{\sigma(1+\epsilon)}{2}\sin 2\theta \qquad (3)$$

B. Superposition of equal and opposite tractions on the boundary of the cavity to make the boundary traction-free.

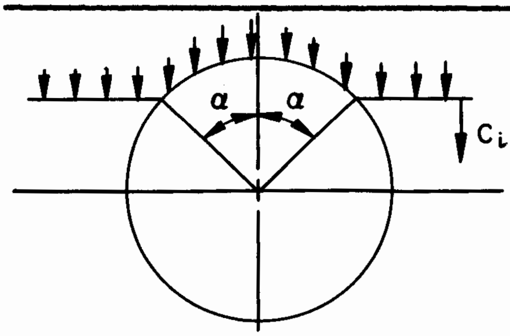


$$\sigma_{rr} = \sigma\left[\cos^2\theta - \epsilon\sin^2\theta\right] \qquad (4)$$

Tension +

$$\sigma_{r\theta} = - \sigma\left(\frac{1+\epsilon}{2}\right)\sin 2\theta \qquad (5)$$

Expansion of σ_{rr} and $\sigma_{r\theta}$ into Fourier Series:



$$\alpha = \cos^{-1}(1 - c_1 t/a) \qquad (6)$$

$$\sigma_{rr} = \frac{a_o}{2}(t) + \sum_{n=1}^{\infty} a_n(t)\cos n\theta \qquad (7)$$

$$\sigma_{r\theta} = \sum_{n=1}^{\infty} b_n(t)\sin n\theta \qquad (8)$$

During Envelopment $t \leq \frac{2a}{c_1}$ where c_1 = velocity of propagation of shock wave across the shelter:

$$a_n = \frac{2}{\pi}\int_0^{\alpha(t)} \sigma_{rr}\cos n\theta \, d\theta \qquad (9)$$

σ_{rr} from Eq. (4)

*$u(t)$ is used to symbolize the unit step function.

$$b_n = \frac{2}{\pi}\int_0^{\alpha(t)} \sigma_{r\theta} \sin n\theta \, d\theta \tag{10}$$

$\sigma_{r\theta}$ from Eq. (5)

$t \leq \frac{2a}{c_i}$ During Envelopment $\qquad \alpha(t) = \cos^{-1}(1 - \frac{c_i t}{a})$

$$\frac{a_o}{2} = \frac{\sigma}{\pi}\left[\frac{\alpha(t)(1-\epsilon)}{2} + \frac{(1+\epsilon)}{4} \sin 2\alpha(t)\right] \tag{11}$$

$$a_n(t) = \frac{2\sigma}{\pi(n+2)}\left[\sin n\alpha \ (\cos^2\alpha - \epsilon \sin^2\alpha) + (\frac{1+\epsilon}{2-n})\sin(2-n)\alpha + (\frac{1-\epsilon}{n})\sin n\alpha\right] \tag{12}$$

$$n \neq 2$$

$$a_2(t) = \frac{2\sigma}{4\pi}\left[(1+\epsilon)\,\alpha + \sin 2\alpha(\cos^2\alpha - \epsilon \sin^2\alpha) + \frac{(1-\epsilon)\sin 2\alpha}{2}\right] \tag{13}$$

$$b_n(t) = -\frac{\sigma}{2\pi}\left[(1+\epsilon)\right]\left[\frac{\sin(2-n)\alpha}{2-n} - \frac{\sin(2+n)\alpha}{2+n}\right] \quad n \neq 2 \tag{14}$$

$$b_2(t) = -\frac{\sigma}{2\pi}(1+\epsilon)\left[\alpha - \frac{\sin 4\alpha}{4}\right] \tag{15}$$

After Envelopment $\qquad t \geq \frac{2a}{c} \quad (\alpha = \pi)$

$$\frac{a_o}{2} = \sigma(\frac{1-\epsilon}{2}) \tag{16}$$

$$a_2(t) = \sigma(\frac{1+\epsilon}{2}) \tag{17}$$

$$a_n(t) = 0 \quad n \neq 2 \tag{18}$$

$$b_2(t) = -\sigma\left(\frac{1+\epsilon}{2}\right) \tag{19}$$

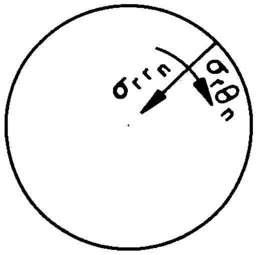
$$b_n(t) = 0 \quad n \neq 2 \tag{20}$$

Curves of the input functions of Eqs. (11) through (20) are shown in Fig. 6.

Equations (11) through (20) serve as input functions for enforcing the condition of zero boundary traction on the cavity surface at r = a. It may be noted that only the modes n = 0 and n = 2 have components $a_o(t)$, $a_2(t)$ and $b_2(t)$ after envelopment time and thus contribute to the long time solution (through poles at zero in the inversion of the integral transform).

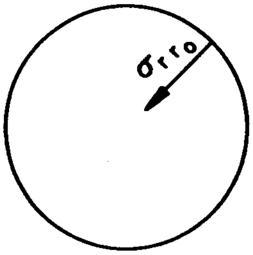
C. Plan of attack: Problem of a cylindrical cavity in an ∞ elastic medium (plane strain).

The assumption of an infinite medium is good for deep cavities during a time of order t = 2 Depth/c_l. The problem is to be solved by superimposing the tractions of Eqs. (11) through (20) on the cavity boundaries. The superposition of these results on the results of the free field with no cavity give the required solution for stresses, velocities, etc. The procedure outlined above leads to the following problems, defined by n, number of circumferential waves in θ direction.



n = n

$$\left.\begin{aligned} \sigma_{rr_n} &= a_n(t)\cos n\theta \\ \sigma_{r\theta_n} &= b_n(t)\sin n\theta \end{aligned}\right\} \tag{21}$$



n = 0

$$\left.\begin{aligned} \sigma_{rr_o} &= a_o(t) \\ \sigma_{r\theta_o} &= 0 \end{aligned}\right\} \tag{22}$$

To facilitate computations and get results which can be applied generally, the inputs will be considered as step pulses:

$$\sigma_{rr_n} = \sigma u(t) \cos n\theta$$

$$\sigma_{r\theta_n} = k \quad \sigma u(t) \sin n\theta \tag{23}$$

$$\sigma_{rr_o} = \sigma u(t) \tag{24}$$

where k is used to indicate the portion of the solution due to the shear, $\sigma_{r\theta_n}$, on the boundary. The results obtained using step functions will serve as influence coefficients; once these influence coefficients are known, the results for any arbitrary function f(t) can be obtained using a Duhamel integral. For example, suppose the hoop stress $\sigma_{\theta\theta_n}(t)$ was desired due to the inputs of Eqs. (11) through (20): If the problems for the step functions were solved,

$$\left\{\begin{aligned} \sigma_{rr_n} &= 1 \cos n\theta \quad u(t) \\ \sigma_{r\theta_n} &= 0 \end{aligned}\right\} \xrightarrow{\text{gives}} \sigma_{\theta\theta_n} = \sigma^*_{\theta\theta_n}(t) \tag{25}$$

$$\left\{\begin{aligned} \sigma_{r\theta_n} &= 1 \sin n\theta \quad u(t) \\ \sigma_{rr_n} &= 0 \end{aligned}\right\} \xrightarrow{\text{gives}} \sigma_{\theta\theta_n} = \sigma^{**}_{\theta\theta_n}(t) \tag{26}$$

Then the result for $\sigma_{\theta\theta_n}$ due to the inputs

$$\left.\begin{aligned} \sigma_{rr_n} &= a_n(t) \cos n\theta \\ \sigma_{r\theta_n} &= b_n(t) \sin n\theta \end{aligned}\right\} \tag{27}$$

is obtained from the Duhamel integral:

$$\sigma_{\theta\theta_n} = \int_o^t \left[\dot{a}_n(\tau)\sigma^*_{\theta\theta_n}(t-\tau) + \dot{b}_n(\tau)\,\sigma^{**}_{\theta\theta_n}(t-\tau) \right] d\tau \tag{28}$$

Other quantities of interest can be obtained in a similar manner.

Quantities of Interest to be investigated include:

1. Hoop stress $\sigma_{\theta\theta}$ at boundary of cavity.
2. Hoop stress $\sigma_{\theta\theta}$ away from boundary.
3. Direct stress (radial) σ_{rr} away from cavity.
4. Velocities, $\dot{w}_n$ and $\dot{v}_n$ of boundary of cavity.
5. Rigid body translation and acceleration in the mode n = 1.

This mode governs the shock loading received by the installations within the cavity.

D. Cylindrical cavity in an ∞ elastic medium, plane strain problem, u = 0, w = w(r, θ, t), v = v(r, θ, t).

The displacement equations of motion may be written as

$$\mu \nabla^2 \mathrm{ul} + (\lambda + \mu)\,\nabla\nabla\cdot \mathrm{ul} = \rho\,\ddot{\mathrm{ul}} \tag{29}$$

where

$$\mathrm{ul} = w(r, \theta, t)\,1k_r + v(r, \theta, t)\,1k_\theta \tag{30}$$

For the Plane Strain Problem,

$$\epsilon_{rr} = w_r,\ \epsilon_{\theta\theta} = \frac{1}{r}\,v_\theta + \frac{w}{r},\ \epsilon_{r\theta} = \frac{1}{2}\left[v_r - \frac{v}{r} + \frac{1}{r}\,w_\theta \right] \tag{31}$$

Note: Subscripts on displacements and potential functions indicate differentiation. For example,

$$u_{rr} = \frac{\partial^2 u}{\partial r^2} \quad \text{etc.}$$

$$\epsilon_{zz} = \epsilon_{z\theta} = \epsilon_{zr} = 0$$

$$\Phi = \lambda\, \theta\, \Phi + 2\mu\, \epsilon \qquad \theta = \nabla \cdot ul$$

$$\left\{\begin{array}{l|l} \sigma_{rr} = \lambda \nabla \cdot ul + 2\mu\epsilon_{rr} & \sigma_{rz} = 0 \\ \sigma_{\theta\theta} = \lambda \nabla \cdot ul + 2\mu\epsilon_{\theta\theta} & \sigma_{\theta z} = 0 \\ \sigma_{r\theta} = 2\mu\, \epsilon_{r\theta} & \sigma_{zz} = \lambda\left[\nabla \cdot ul\right] \end{array}\right\} \tag{32}$$

We define two potential functions, ϕ (r, Θ, t) and ψ (r, Θ, t) such that

$$w = \phi_r + \frac{1}{r}\psi_\theta \tag{33}$$

$$v = \frac{1}{r}\phi_\theta - \psi_r \tag{34}$$

$$u = 0 \tag{35}$$

where:

$$c_1^2 \nabla^2\phi = \ddot{\phi} \tag{36}$$

$$c_2^2 \nabla^2\psi = \ddot{\psi} \tag{37}$$

and

$$c_1^2 = \frac{\lambda + 2\mu}{\rho},\ c_2^2 = \frac{\mu}{\rho} \tag{38}$$

It may be noted that the dilitation, $\nabla \cdot ul = \nabla^2\phi$ and the rotation

$$\nabla \times ul = -\nabla^2\psi \tag{39}$$

Using a transform in time:

$$\bar{f}(r, \theta, \Omega) = \frac{1}{2\pi}\int_0^\infty f(r, \theta, t)e^{-i\Omega t}\, dt \tag{40}$$

$$f(r, \theta, t) = \int_{-\infty - i\gamma}^{\infty - i\gamma} \bar{f}(r, \theta, \Omega)e^{i\Omega t}\, d\Omega \tag{41}$$

and letting

$$\bar{\phi}(r, \theta, \Omega) = \bar{\phi}\,(r, \Omega)\cos n\theta \tag{42}$$

$$\bar{\psi}(r,\theta,\Omega) = \bar{\psi}\,(r,\Omega)\ \sin n\theta \tag{43}$$

Eqs. (36) and (37) become

$$\bar{\phi}_{rr} + \frac{1}{r}\bar{\phi}_r + \left(\frac{\Omega^2}{C_1^2} - \frac{n^2}{r^2}\right)\bar{\phi} = 0 \tag{44}$$

$$\bar{\psi}_{rr} + \frac{1}{r}\bar{\psi}_r + \left(\frac{\Omega^2}{C_2^2} - \frac{n^2}{r^2}\right)\bar{\psi} = 0 \tag{45}$$

For outgoing waves Note: $H_n^{(2)}\ (\zeta r) \longrightarrow e^{-i\zeta r}$ as $\zeta r >>>$.

$$\bar{\phi}(r,\theta,\Omega) = A_n H_n^{(2)}\left(\frac{\Omega r}{C_1}\right)\cos n\theta \tag{46}$$

and

$$\bar{\psi}(r,\theta,\Omega) = B_n H_n^{(2)}\left(\frac{\Omega r}{c_2}\right)\sin n\theta \tag{47}$$

where $H_n^{(2)}$ is the Hankel Function of the Second Kind of Order n. The coefficients A_n and B_n are evaluated from the boundary conditions of the applied tractions σ_{rr} and $\sigma_{r\theta}$ on the boundary on the cavity.

Determination of the Boundary Conditions

The stresses σ_{rr} and $\sigma_{r\theta}$ are given below in terms of ϕ and ψ:

$$\sigma_{rr} = \lambda\theta + 2\mu\epsilon_{rr} = \lambda\nabla^2\phi + 2\mu\left[\phi_{rr} - \frac{1}{r^2}\psi_\theta + \frac{1}{r}\psi_{r\theta}\right] \tag{48}$$

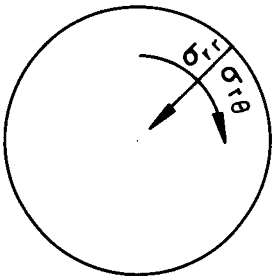
$$\sigma_{r\theta} = 2\mu\epsilon_{r\theta} = \mu\left[\frac{2}{r}\phi_{r\theta} - \frac{2}{r^2}\phi_\theta + \frac{1}{r}\psi_r - \psi_{rr} + \frac{1}{r^2}\psi_{\theta\theta}\right] \tag{49}$$

Using the notation of Eqs. (42) and (43) and applying the transform Eq. (40),

$$\frac{\bar{\sigma}_{rr}}{\cos n\theta} = (\lambda + 2\mu)\,\bar{\phi}_{rr} + \lambda\left[\frac{1}{r}\bar{\phi}_r - \frac{n^2}{r^2}\bar{\phi}\right] + 2\mu\left[\frac{n}{r}\bar{\psi}_r - \frac{n}{r^2}\bar{\psi}\right] \tag{50}$$

$$\frac{\bar{\sigma}_{r\theta}}{\sin n\theta} = \mu\left[-\frac{2n}{r}\bar{\phi}_r + \frac{2n}{r^2}\bar{\phi} + \frac{1}{r}\bar{\psi}_r - \bar{\psi}_{rr} - \frac{n^2}{r^2}\bar{\psi}\right] \tag{51}$$

The boundary conditions become:



$$\left.\begin{aligned} \sigma_{rr}\Big]_{r=a} &= \sigma\, u(t)\cos n\theta \\ \sigma_{r\theta}\Big]_{r=a} &= k\,\sigma\, u(t)\sin n\theta \end{aligned}\right\} \tag{52}$$

or, applying Eq. (40),

$$\bar{\sigma}_{rr}\Big]_{r=a} = \frac{\sigma\cos n\theta}{2\pi i\Omega} \tag{53}$$

$$\bar{\sigma}_{r\theta}\Big]_{r=a} = \frac{k\sigma\sin n\theta}{2\pi i\Omega} \tag{54}$$

Using Eqs. (46), (47), (50) and (51), and substituting into Eqs. (53) and (54), the coefficients A_n and B_n can be evaluated as:

$$A_n = \frac{\dfrac{\sigma a^2}{2\pi i\Omega\mu}B + \dfrac{k\sigma a^2}{2\pi i\Omega\mu}E}{FB + DE} \tag{55}$$

$$B_n = \frac{-\dfrac{\sigma a^2}{2\pi i\Omega\mu}D + \dfrac{k\sigma a^2}{2\pi i\Omega\mu}F}{FB + DE} \tag{56}$$

where

$$F = H_n^{(2)}(\xi)\left[2n(n+1) - 3\xi^2\right] - 2\xi H_{n-1}^{(2)}(\xi) \tag{57}$$

$$E = H_n^{(2)}\left(\frac{c_1}{c_2}\xi\right)\left[2n(1+n)\right] - 2n\frac{c_1}{c_2}\xi H_{n-1}^{(2)}\left(\frac{c_1}{c_2}\xi\right) \tag{58}$$

$$D = H_n^{(2)}(\xi)\left[2n(n+1)\right] - 2n\xi H_{n-1}^{(2)}(\xi) \tag{59}$$

$$B = H_n^{(2)}\left(\frac{c_1}{c_2}\xi\right)\left[-2n(n+1) + \frac{c_1^2}{c_2^2}\xi^2\right] + 2\frac{c_1}{c_2}\xi H_{n-1}^{(2)}\left(\frac{c_1}{c_2}\xi\right) \tag{60}$$

and $\bar{\phi}_n(r,\theta,\Omega)$ and $\bar{\psi}_n(r,\theta,\Omega)$ are given by Eqs. (46) and (47). It should be noted that when terms involving k are set = 0 and $\sigma = 1$, the problem defined by Eq. (25) is considered; when terms not involving k are set = 0 and $k\sigma = 1$, the problem defined by Eq. (26) is considered.

Determination of the Hoop Stress $\sigma_{\theta\theta_n}$ for any n

Noting that

$$\bar{\sigma} = \lambda\bar{\theta}\phi + 2\mu\bar{\epsilon} = \lambda\nabla^2\bar{\phi} + 2\mu\overline{e_{\theta\theta}} \tag{61}$$

and

$$\frac{\bar{\sigma}_{\theta\theta}}{\cos n\theta} = \lambda\left[\bar{\phi}_{rr} + \frac{1}{r}\bar{\phi}_r - \frac{n^2}{r^2}\bar{\phi}\right] + 2\mu\left[-\frac{n^2}{r^2}\bar{\phi} - \frac{n}{r}\overline{\psi_r} + \frac{1}{r}\overline{\phi_r} + \frac{n}{r^2}\overline{\psi}\right] \tag{62}$$

For $\lambda = \mu$ i.e., $\gamma = 1/4$

$$\bar{\sigma}_{\theta\theta} = \mu\cos n\theta\left[\frac{3}{r}\overline{\phi_r} + \bar{\phi}_{rr} - \frac{3n^2}{r^2}\bar{\phi} - \frac{2n}{r}\overline{\psi_r} + \frac{2n}{r^2}\overline{\psi}\right] \tag{63}$$

Eq. (63) upon substitution becomes:

$$\frac{\overline{\sigma\theta\theta_n}}{\sigma} = \frac{\cos n\theta}{2\pi i\Omega}\frac{\left\{\begin{array}{l}(B + kE)\left[\frac{3a}{r}\xi H_n^{(2)'}\left(\frac{\xi r}{a}\right) + \xi^2 H_n^{(2)''}\left(\xi\frac{r}{a}\right) - \frac{3n^2a^2}{r^2}H_n^{(2)}\left(\xi\frac{r}{a}\right)\right] \\ \quad + (kF-D)\left[\frac{2na^2}{r^2}H_n^{(2)}\left(\xi\frac{c_1}{c_2}\frac{r}{a}\right) - \frac{2na}{r}\frac{c_1}{c_2}\xi H_n^{(2)'}\left(\xi\frac{c_1}{c_2}\frac{r}{a}\right)\right]\end{array}\right\}}{FB + DE}$$

$$= \frac{\cos n\theta}{2\pi i\Omega} G_n(r,\xi) \tag{64}$$

where all derivatives are taken with respect to their argument. The value of $\sigma_{\theta\theta_n}$ is then obtained by the use of the inversion integral:

$$\frac{\sigma_{\theta\theta_n}(r,\theta,t)}{\sigma} = \frac{1}{2\pi i}\int_{-\infty-i\gamma}^{\infty-i\gamma}\frac{G_n(r,\xi)}{\xi}e^{i\xi ct/a}\,d\xi\,\cos n\theta \tag{65}$$

The inversion integral of Eq. (65) contains two parameters, r the distance out from the center of the cavity and t, the time.

For the case of $\left.\sigma_{\theta\theta}\right]_{r=a}$, the hoop stress at the boundary of the tunnel is given by

$$\frac{\bar{\sigma}_{\theta\theta_n}}{\sigma} = \frac{\cos n\theta}{2\pi i\Omega}\ \frac{\left\{\begin{array}{l}(B+kE)\left[(-2n^2 - 2n - \xi^2)\, H_n^{(2)}(\xi) + 2\xi H_{n-1}^{(2)}(\xi)\right] \\ + (kF-D)\left[2n(n+1)\, H_n^{(2)}\left(\frac{c_1}{c_2}\xi\right) - 2n\frac{c_1}{c_2}\xi\, H_{n-1}^{(2)}\left(\frac{c_1}{c_2}\xi\right)\right]\end{array}\right\}}{\left[FB + DE\right]} \quad (66)$$

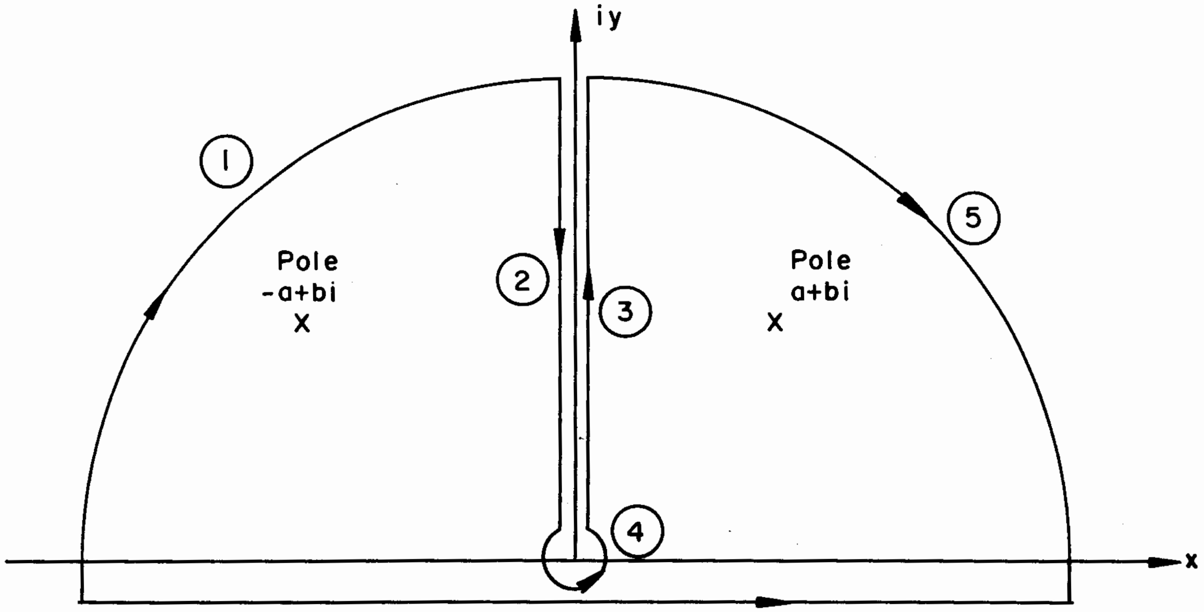
and

$$\frac{\sigma_{\theta\theta_n}}{\sigma} = \frac{1}{2\pi i}\int_{-\infty-i\gamma}^{\infty-i\gamma} \frac{\left\{\begin{array}{l}(B+kE)\left[(-2n^2-2n-\xi^2)\, H_n^{(2)}(\xi) + 2\xi H_{n-1}^{(2)}(\xi)\right] \\ + (kF-D)\left[2n(n+1)\, H_n^{2}\left(\frac{c_1}{c_2}\xi\right) - 2n\frac{c_1}{c_2}\xi\, H_{n-1}^{(2)}\left(\frac{c_1}{c_2}\xi\right)\right]\end{array}\right\}}{\xi\left[FB + DE\right]} e^{i\xi ct/a}\, d\xi \cos n\theta \quad (67)$$

In addition to hoop stresses at and in the neighborhood of the cavity, the radial stress σ_{rr} and the velocity field are also of interest.

Inversion of Integral--Eq. (67)

The integral of Eq. (67) is inverted over the following contour:



Some General Remarks on the Inversion Integral

1. Integral on Paths (1) and (5) $\longrightarrow 0$ as $R \longrightarrow \infty$ for all n.

2. Branch Integral--the integration over Paths (2) and (3), has been set up for any n. This requires numerical integration by computer. Work is proceding on n = 1, 2.

3. Point $\xi = 0$ is both a branch point and a pole, in general. The integral over Path (4) will thus contribute an amount to $\sigma_{\theta\theta_n}$, the value being dependent on the pole at $\xi = 0$. It should be noted however, that when the results of the integration of Eq. (67) are used as influence coefficients to obtain the response due to the input of Eqs. (11) through (20), only the poles at zero in the modes n = 0 and n = 2 remain and contribute to the long term ($t \rightarrow \infty$) solution for $\sigma_{\theta\theta}$.

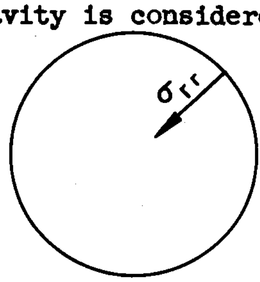
4. In general

$$\sigma_{\theta\theta_n}(r, \theta, t) = I_2 + I_3 + I_4 + 2\pi i \sum \text{Residues} \tag{68}$$

where the residues are evaluated for the complex poles in the upper portion of the plane.

Cylindrical Cavity in an ∞ Elastic Medium, n = 0

The cavity is considered under a symmetric pressure loading given below:



$$\left.\begin{aligned} \sigma_{rr_o} &= \sigma u(t) \\ \sigma_{r\theta_o} &= 0 \end{aligned}\right\} \text{Symmetric problem} \tag{69}$$

We define a potential function $\phi(r, t)$ such that

$$w = \phi_r, \; u = v = 0 \tag{70}$$

Proceeding as in the case $n \neq 0$

$$\phi(r, \Omega) = A_o H_o^{(2)} \left(\frac{\Omega r}{c_1}\right) \tag{71}$$

where

$$A_o = -\frac{\sigma a^3}{4\pi i \mu c_1 \xi^2} \left[\frac{1}{1.5\xi H_o^{(2)}(\xi) - H_1^{(2)}(\xi)} \right] \tag{72}$$

Inversion Integral for σ_{rr_o} at any point in the field

The transformed radial stress $\bar{\sigma}_{rr_o}$ becomes

$$\bar{\sigma}_{rr_o} = \lambda \nabla^2 \bar{\phi} + 2\mu \bar{\phi}_{rr} = \frac{\sigma}{2\pi i \Omega} \left[\frac{3\xi H_o^{(2)}(\frac{\Omega r}{c_1}) - \frac{2a}{r} H_1^{(2)}(\frac{\Omega r}{c_1})}{3\xi H_o^{(2)}(\xi) - 2H_1^{(2)}(\xi)} \right] \tag{73}$$

and the stress σ_{rr} is obtained by means of the inversion integral

$$\sigma_{rr_o}(r, t) = \int_{-\infty - i\gamma}^{\infty - i\gamma} \frac{\sigma}{2\pi i \xi} \left[\frac{3\xi H_o^{(2)}(\frac{\xi r}{a}) - \frac{2a}{r} H_1^{(2)}(\frac{\xi r}{a})}{3\xi H_o^{(2)}(\xi) - 2H_1^{(2)}(\xi)} \right] e^{\frac{(i\xi c_1 t)}{a}} d\xi \tag{74}$$

Inversion Integral for $\sigma_{\theta\theta_o}$ at any point in field

The transformed hoop stress $\bar{\sigma}_{\theta\theta_o}$ becomes

$$\bar{\sigma}_{\theta\theta_o} = (\lambda + 2\mu) \frac{\bar{\phi}_r}{r} + \lambda \bar{\phi}_{rr} = \mu \left[3 \frac{\bar{\phi}_r}{r} + \bar{\phi}_{rr} \right] \tag{75a}$$

$$= \frac{\sigma a}{4\pi i c_1} \left[\frac{\frac{2a}{r} H_1^{(2)}(\frac{\Omega r}{c_1}) + \xi H_o^{(2)}(\frac{\Omega r}{c_1})}{\xi \left[1.5\xi H_o^{(2)}(\xi) - H_1^{(2)}(\xi) \right]} \right] \tag{75b}$$

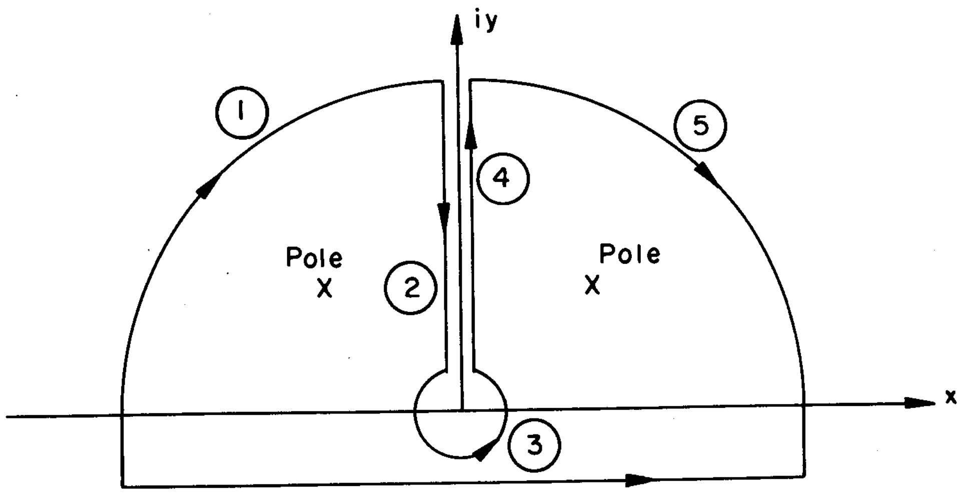
and the hoop stress $\sigma_{\theta\theta}$ is obtained by means of the inversion integral

$$\sigma_{\theta\theta_o}(r, t) = \int_{-\infty - i\gamma}^{\infty - i\gamma} \frac{\sigma}{4\pi i} \left[\frac{2 \frac{a}{r} H_1^{(2)}(\frac{\Omega r}{c_1}) + \xi H_o^{(2)}(\frac{\Omega r}{c_1})}{\xi \left[1.5\xi H_o^{(2)}(\xi) - H_1^{(2)}(\xi) \right]} \right] e^{\frac{i\xi c_1 t}{a}} d\xi \tag{76}$$

Equation (76) will be inverted for the case r=a, i.e., the hoop stress $\sigma_{\theta\theta_0}$ (t) on the boundary of the cavity:

$$\sigma_{\theta\theta_0}(t) = \int_{-\infty-i\gamma}^{\infty-i\gamma} \frac{1}{4\pi i} \left[\frac{2H_1^{(2)}(\frac{\Omega r}{c_1}) + \xi H_o^{(2)}(\frac{\Omega r}{c_1})}{\xi(1.5\xi H_o^{(2)}(\xi) - H_1^{(2)}(\xi)\ )} \right]_{r=a} e^{i\xi c_1 t/a}\, d\xi \tag{77}$$

Evaluation of the Inversion Integral



$$\sigma_{\theta\theta_0}(t) = I_1 + I_2 + I_3 + I_4 + I_5 + 2\pi i \sum_{\text{Poles in interior}} \text{Residues} \tag{78}$$

Note: ξ = 0 is both a branch point and a pole.

$$I_{(2)} + I_{(4)} = -2\sigma \int_0^\infty \frac{e^{-uct/a}\, du}{u\left[K_1(u) + 1.5uK_o(u)\right]^2 + \frac{\pi^2 u}{4}\left[3uI_o(u) - 2I_1(u)\right]^2} \tag{79}$$

$I_3 = -\sigma$, this is the long time solution due to pole at u = 0. (80)

$I_1 = I_5 = 0$ as $R \rightarrow \infty$.

The denominator of Eq. (79) yields the two simple poles λ_1 and λ_2:

Simple poles at $\lambda_1 = .4464 + .4410i$

$\lambda_2 = .4464 - 4410i$

and

$$2\pi i \sum_{\lambda=\lambda_1\lambda_2} \text{Residues} = \left\{ \sigma \left[\frac{2H_1^{(2)}(\lambda) + \lambda H_o^{(2)}(\lambda)}{\lambda\left[4H_o^{(2)}(\lambda) - 3\lambda H_1^{(2)}(\lambda)\right]} \right] e^{i\lambda c_1 t/a} \right\}_{\lambda=\lambda_1\lambda_2} \tag{81}$$

$$= \sigma \left[1.6812\cos(.4464\,\frac{c_1 t}{a}) - .7444\,\sin(.4464\,\frac{c_1 t}{a}) \right] e^{-.4410\frac{c_1 t}{a}}$$

The stress $\sigma_{\theta\theta_o}$ at r = a may be computed from the relation

$$\sigma_{\theta\theta_o}\Big]_{r=a} = \text{Eq. (79) plus Eq. (80) plus Eq. (81)} \tag{82}$$

and represents the hoop compression due to the applied hoop tension $\sigma u(t)$ over the interior boundary of the cavity. The result is plotted in Fig. 7.

From Eqs. (11) and (16), the time dependency of the applied tension $\frac{a_o}{2}(t)$ is

$$\frac{a_o}{2}(t) = \frac{\sigma}{\pi}\left[(\frac{1-\epsilon}{2})\cos^{-1}(1 - \frac{ct}{a}) + (\frac{1+\epsilon}{2})(1 - \frac{ct}{a})\sqrt{2\frac{ct}{a} - \frac{c^2t^2}{a^2}} \right] \qquad t \leq \frac{2a}{c} \tag{83}$$

$$= \sigma\,(\frac{1-\epsilon}{2}) \qquad t \geq \frac{2a}{c}$$

As an example, the particular case where $\epsilon = 0$, will be considered. The value of the time derivative of the input function $a_o(t)$ is given by

$$\frac{a_o}{2}(t) = \frac{\sigma c_1}{2\pi a}\left[\frac{2 - 4\,\frac{c_1 t}{a} + 2\,\frac{c_1^2 t^2}{a^2}}{\sqrt{2\,\frac{c_1 t}{a} - \frac{c_1^2 t^2}{a^2}}} \right] \tag{84}$$

Setting $\sigma = 1$, in Eq. (82) and denoting those results as $\sigma^*_{\theta\theta_o}$, the stress $\sigma_{\theta\theta_o}(t)$ is obtained from the Duhamel integral:

$$\sigma_{\theta\theta_o}(t) = \int_o^t \frac{\dot{a}_o}{2}(\tau)\, \sigma^*_{\theta\theta_o}(t - \tau)\, d\tau \qquad (85)$$

The results obtained from Eq. (85) are plotted in Fig. 8 for the case n = 0. For long times, $t \longrightarrow \infty$, the stress $\sigma_{\theta\theta_o}(t)$ becomes a hoop compression of magnitude $-\frac{\sigma}{2}$.

General Remarks--Work in Progress

1. n = 0. The velocity, w_o, and pressure $\sigma_{\theta\theta_o}$, responses of the cavity boundary have been evaluated for the pressure loading $\frac{a_o}{2}(t)$.

2. n = 1, 2, ...n. The contribution of $\sigma_{\theta\theta_n}$ of the branch integral over paths (2) and (3) has been set up in general terms for any n and is a real integral of the modified Bessel Functions I_n and K_n. This integral can easily be evaluated by numerical integration on a computer and work has been started for the modes n = 1, 2. The problem of the determination of the poles of FB + DE for n = 1, 2, ... is under study. Rather complex mapping procedures with available tables of $H_n^{(2)}(z)$ for complex arguments are required. The poles for n = 1 and their corresponding residues have been determined (See Fig. 5). Work is proceeding for the mode n = 2 which contributes a part of the long term solution ($t \rightarrow \infty$) of the problem.

3. In addition, the accelerations on the mode n = 1, due to the rigid body translation of the cavity, $w - v_\theta$, are also being determined; these results will give the major shock effects felt by the installations in the cavity.

It is hoped in view of the long time pressure history associated with large yield weapons, that the essential contribution to the response of the shelter will be determined from the lower modes of n, $n = 0, 1, 2, 3$. This cannot be definitely ascertained until the responses in the aforementioned modes are obtained. If the problem should turn out to be quasi-static, i.e., the stresses at long times control, considerable simplifications can be made in the analysis. On the other hand, if relatively short time stresses should control and higher modes are of importance, it is hoped that the response for these higher modes will be obtained using short time asymptotic techniques in the inversion integrals for $n \geq 3$.

BIBLIOGRAPHY

(Underground Phenomenology Session)

This bibliography concerns itself with certain topics which are pertinent to theoretical investigations of the response and failure mechanism of deep underground cavities subjected to a large-yield nuclear surface burst. It is far from complete, although only very few papers were found which are specifically directed at the problem of interest. Brief comments on some papers which are especially important are included.

A. FREE FIELD PHENOMENA

The following are standard works which include discussions on wave propagation:

1. Kolsky, H., Stress Waves in Solids, Oxford University Press, 1953.

2. Lamb, H., Hydrodynamics, Dover, New York.

3. Love, H. E. H., A Treatise on the Mathematical Theory of Elasticity, Dover, New York.

4. Courant, R. and K. D. Friedrichs, Supersonic Flow and Shock Waves, Interscience, New York, 1948. (of special interest, Chapter III E and Appendix)

Works which are specifically concerned with wave propagation due to various types of surface or interior loading:

5. Broberg, K. B., Shock Waves in Elastic and Elastic-Plastic Media, Stockholm, 1956.

6. Lang, H. A., Surface Displacements In An Elastic Half Space, The RAND Corporation, Paper P-1498, April 1, 1958.

7. Huth, J. H., Estimating Ground Motions Resulting From Air-Induced Ground Shocks, The RAND Corporation, Research Memorandum RM-1762, July, 1956.

8. Cole, H. D., and J. H. Huth, Elastic Stresses Produced in a Half Plane by Steadily Moving Loads, The RAND Corporation, Paper P-884, June 1956.

9. Kochina, N. N., and Melnikova, N. S. "Strong Point Blast in a Compressible Medium," Prikladnaia Matematika i Mehanika, V. 2, 1958.

10. Pekeris, L. L., "The Seismic Surface Pulse," Proc. National Academy of Science, Vol. 41.7, pp. 469-480, 1955.

11. ______________, "The Seismic Buried Pulse" ______________ Vol. 41.9, pp. 629-639, 1955.

12. ______________, "Solutions of an Integral Equation Occurring in Impulsive Wave Propagation Problems," ______________, Vol. 42, pp. 439-443, 1956.

13. ______________ and H. Lifson, "Motion of the Surface of a Uniform Elastic Half-Space Produced by a Buried Pulse," Journal of Acoustical Soc. of America, Vol. 29, 1957.

14. Chao, Chi-chang, "Dynamical Response of an Elastic Half-Space to Tangential Surface Loadings," Institute of Flight Structures, Columbia University, New York, ONR Proj. NR 064-401.

15. Mindlin, R. D., "Force at a Point in the Interior of a Semi-Infinite Solid," Physics, Vol. 7, 1936.

16. Sauter, F., "Der Elastische Halbraum be ieiner Mechanischen Beinfluissung Seiner Oberflaeche," Zeitschr. Angew. Math. und Mech., pp. 94-95, 1950.

In the above list Broberg's paper (5) is a fairly complete summary of the whole problem of free field phenomena and of the interaction and failure modes of cavities. Free field solutions are discussed for impulsive surface loads, extensive numerical data are given for the case of an impulsive line load, and some data for the case of a concentrated load. The papers by Lang, Pekeris and Chao give various types of closed form solutions, which may be expanded by numerical integrations to obtain the solution for an expanding surface pulse.

B. CAVITIES IN AN ELASTIC MEDIUM

Investigations dealing with spherical and cylindrical waves of interest to studies concerned with the response of cavities:

17. Nishimana, G., "On the Elastic Waves Due to Pressure Variation on the Inner Surface of a Spherical Cavity in An Elastic Solid," Bulletin Earthquake Research Institute, Tokyo, Vol. 15, 1937, p. 614.

18. Selberg, H. L., "Transient Compression Waves From Spherical and Cylindrical Cavities," Arkiv for Fysik, Stockholm, 1952, Vol. 5, pp. 97-108.

19. Sezawa, K., "Dilatational and Distortional Waves Generated from a Cylindrical or a Spherical Origin."

20. Weibull, W., "Boundary Conditions at the Surface of a Detonating Charge of High Explosive," Arkive for Matematik, Astronomi och Fysik, 34 B, 1947.

21. Vanek, J., "A Contribution to the Theory of Elastic Waves Produced by Shock," Czechoslavak Journal of Physics, Vol. 3, 1953, pp. 97-117.

While the results given in the above papers are of great interest, the solutions are restricted to the mode n = 0, with exception of Vanek's work which also includes contributions due to the mode n = 1, in case of a spherical cavity subjected to a forcing function of the form $\sigma_o \alpha^{\nu} t^{\nu} e^{-\alpha t}$. The more pertinent case of a cylindrical cavity has so far apparently not been studied for modes higher than n = 0, but preliminary investigations show that significant contributions may be expected in the higher modes.

C. EXPERIMENTAL AND THEORETICAL WORK ON TUNNEL FAILURES AND STRESSES AROUND TUNNELS

A series of experiments with HE on small scale tunnels and by means of photoelastic methods have been carried out in the last decade.

22. O.C.E., Eng. Dir., Protective Construction Branch: Final Reports--Underground Explosion Tests at Dugway 1950-52.

23. Colorado School of Mines: Report of Test Investigation of a Model Tunnel Section Subjected to Scaled Detonations, June 1949.

24. ____________, Report of Test-Photoelastic or Other Investigation of Model Tunnel Sections Subjected to Impulsive Loading.

25. ____________, "Symposium on Rock Mechanics," Quarterly of the Colorado School of Mines, Vol. 51, No. 3, 1956.

26. ____________, "An Introduction to the Design of Underground Openings for Defense," Quarterly of the Colorado School of Mines, Vol. 46, No. 1, 1951.

27. Lampson, C. W., Final Report on Effects of Underground Explosions, NRDC Report A-479 OSRD Rep. 6645, Feb., 1946.

28. Johansson, C. H., "Plastic Deformation and the Formation of Cracks by Detonating Charges," IVA, Periodical of Sci. Engr. Research, The Royal Swedish Ac. of Eng. Sciences, Vol. 26, 1955, pp. 16-29.

Considerable literature exists regarding a static stress field around cavities in solids, effects of stress concentrations, etc., which are treated in many texts on elasticity (e.g. Timoshenko: Theory of Elasticity). Two papers by Hiltscher, concerning the static case, are of interest because they discuss the effect of the shape of the openings under both uniaxial and biaxial stress fields:

29. Hiltscher, R., "Die Totale Zugkraft an Oeffnungen in einem einachsigen Druckspannungsfeld," Der Bauingenieur, 32, 1957, Heft 12.

30. ___________, "Spannungen an Tunnel Oeffnunger mit rechteckigem Nuz querschnitt und kreisbogenformiger Ueberwoelbung," Der Bauingenieur, 32, 1957, Heft 8.

The problem of tunnel lining can be treated as that of an elastic boundary in an elastic medium. This problem has so far been solved for an acoustic medium only in Baron's paper:

31. Baron, M. L., "Response of Nonlinearly Supported Spherical Boundaries to Shock Waves," Journal of Applied Mechanics, No. 57-APM12, 1957.

No specific work on the dynamic failure mechanism around cavities seems to be avilable. Broberg's paper (listed under free field phenomena) contains a summary of the experimental and the theoretical work. General treatment of the problem is contained in the following works:

32. Bridgman, P. W., The Physics of High Pressure, London, 1931.

33. ___________, Studies in Large Plastic Flow and Fracture, McGraw-Hill, 1952.

34. Freudenthal, F., The Inelastic Behavior of Engineering Materials and Structure, Wiley, 1950.

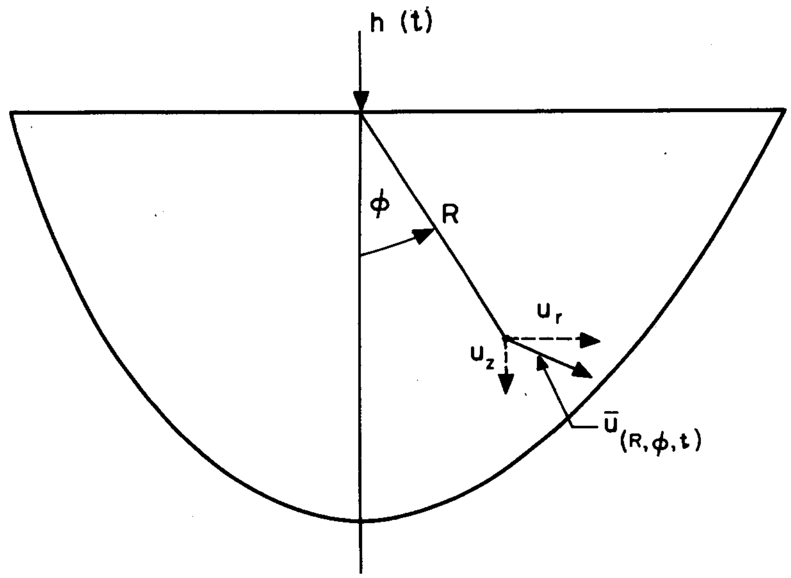


Fig. 1

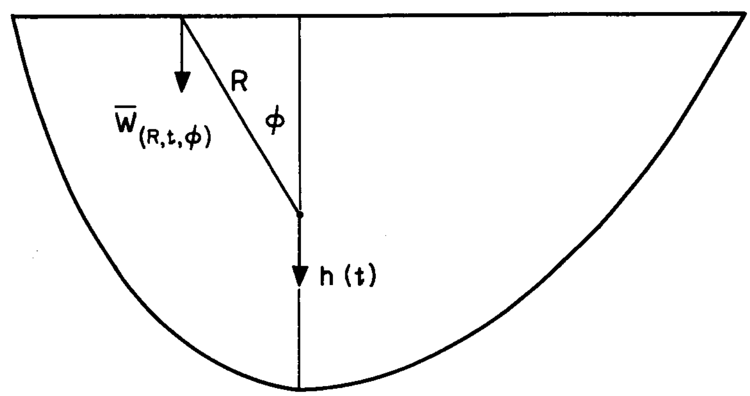


Fig. 2

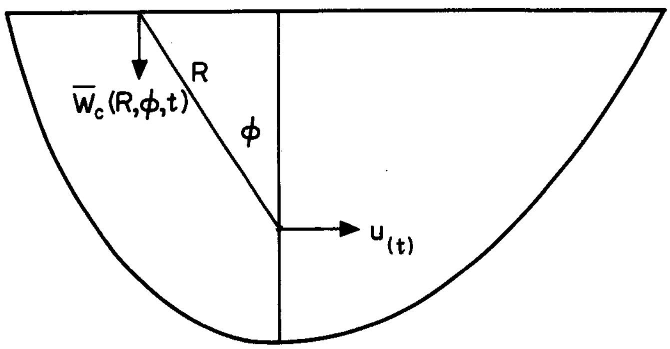

R
$\overline{W}_c(R,\phi,t)$
ϕ
$u_{(t)}$

Fig. 3

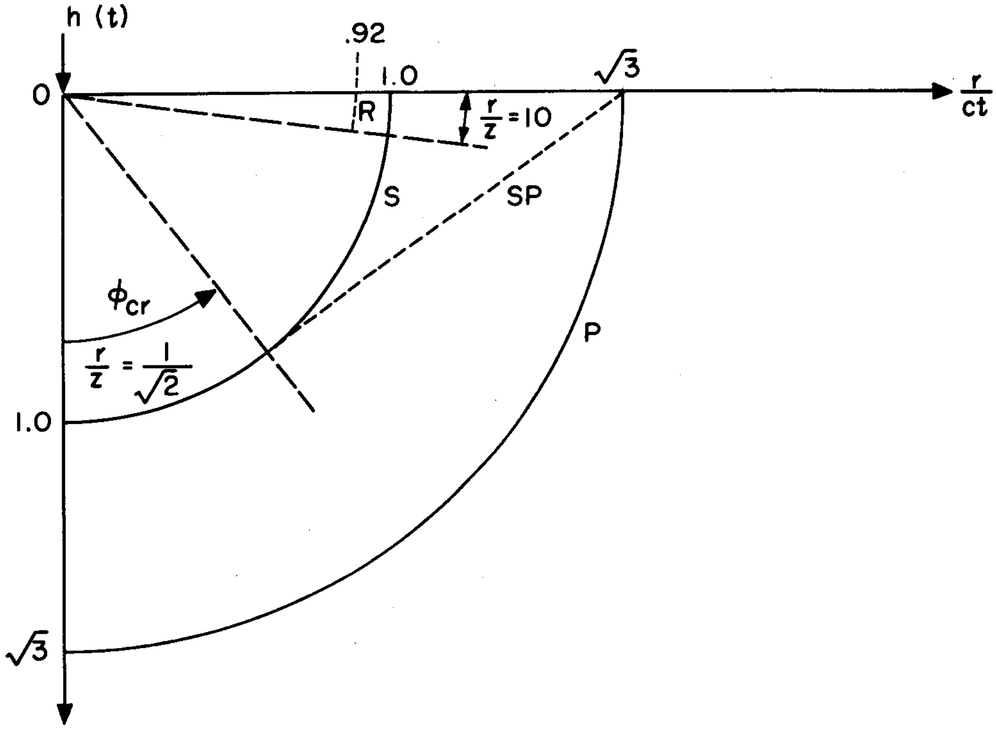

h (t)
.92
1.0
$\sqrt{3}$
$\frac{r}{ct}$
0
R
$\frac{r}{z}=10$
S
SP
ϕ_{cr}
$\frac{r}{z}=\frac{1}{\sqrt{2}}$
P
1.0
$\sqrt{3}$

Fig. 4

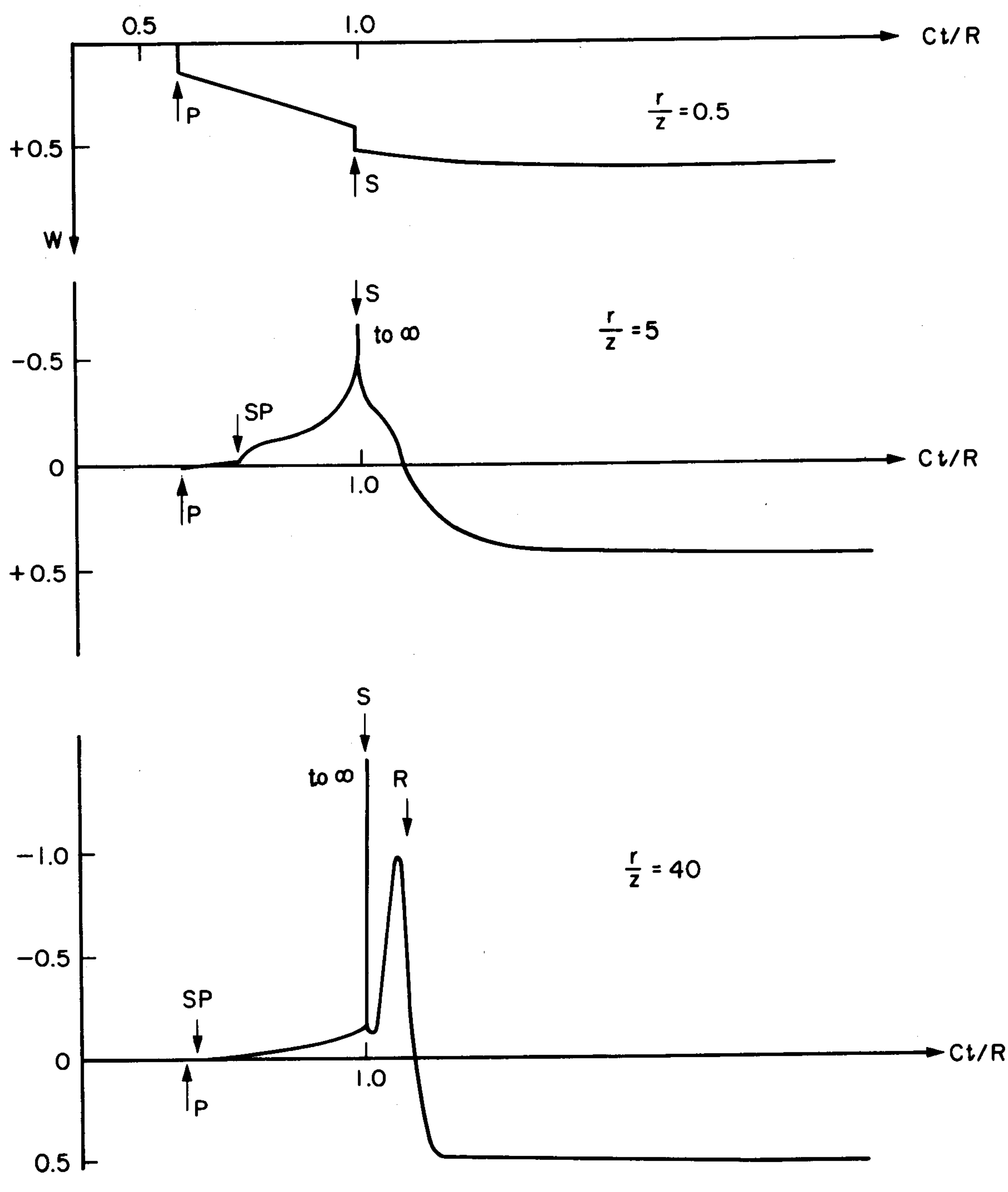


Fig. 5 —— $W = \frac{1}{3}\ u\,\pi^2 R u_z$ positive values mean downward displacements

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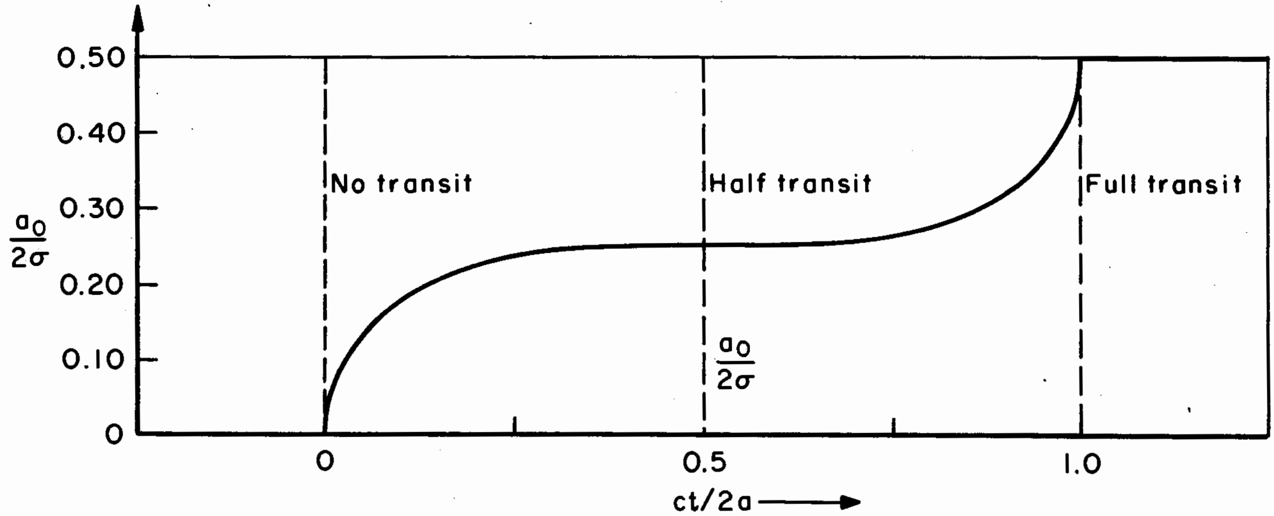
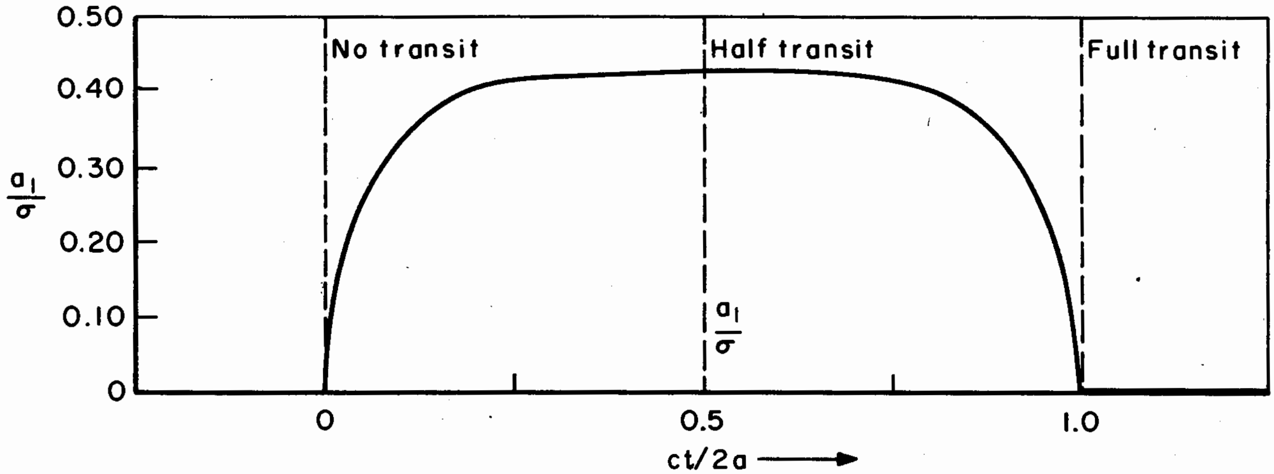




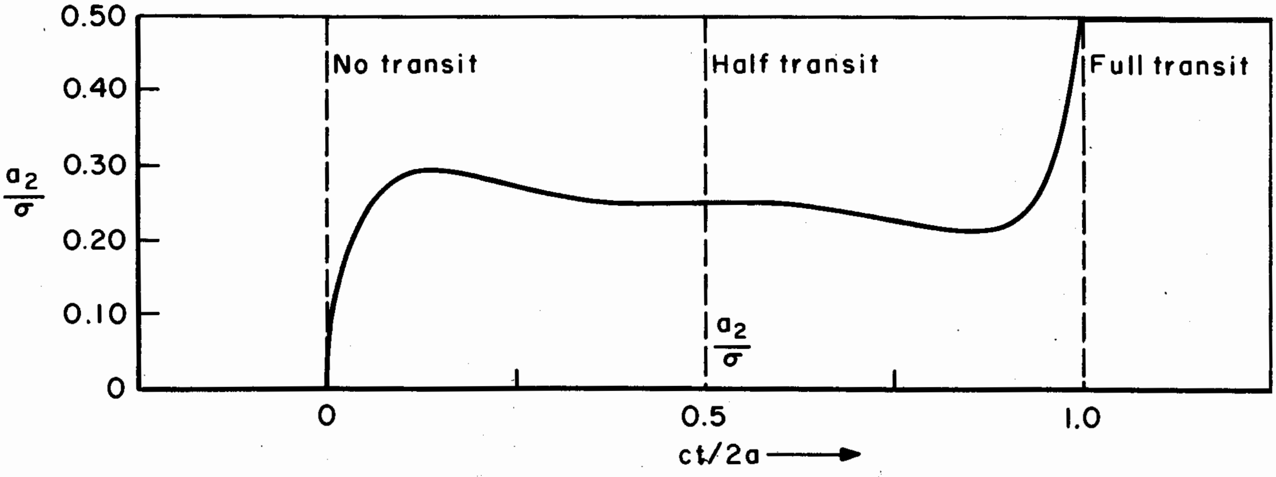
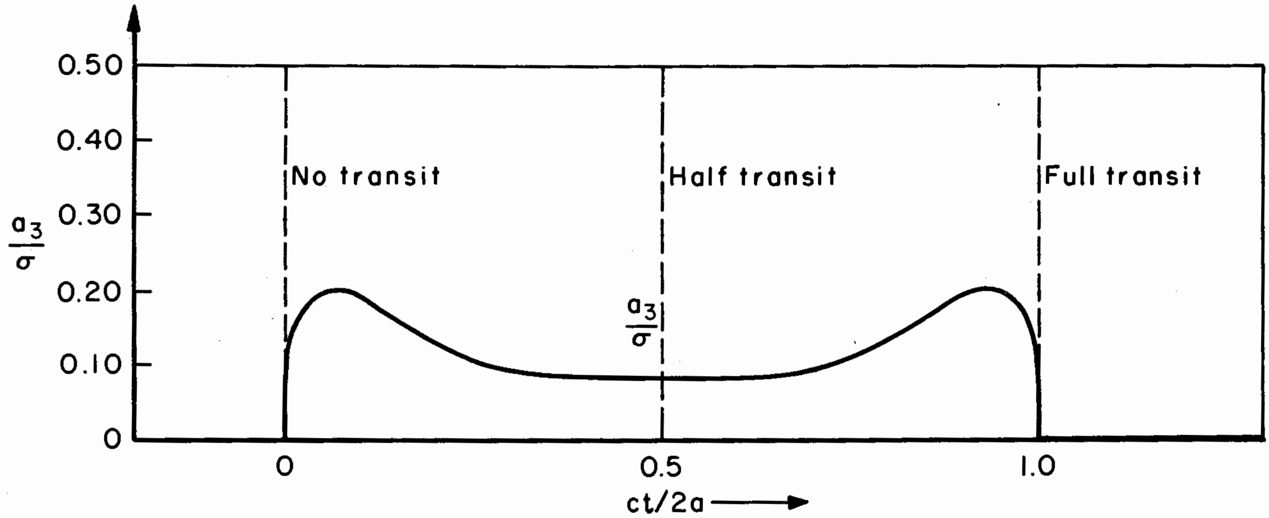


Fig.6—Input pressures, $\epsilon = 0$



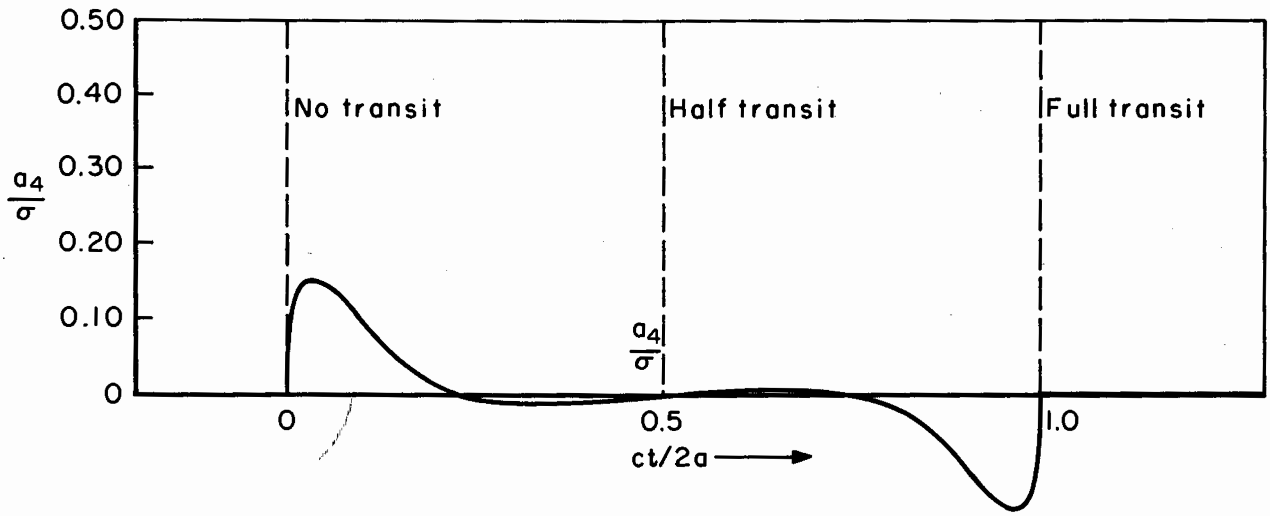


Fig. 6 (Continued)—Input pressures, $\epsilon = 0$

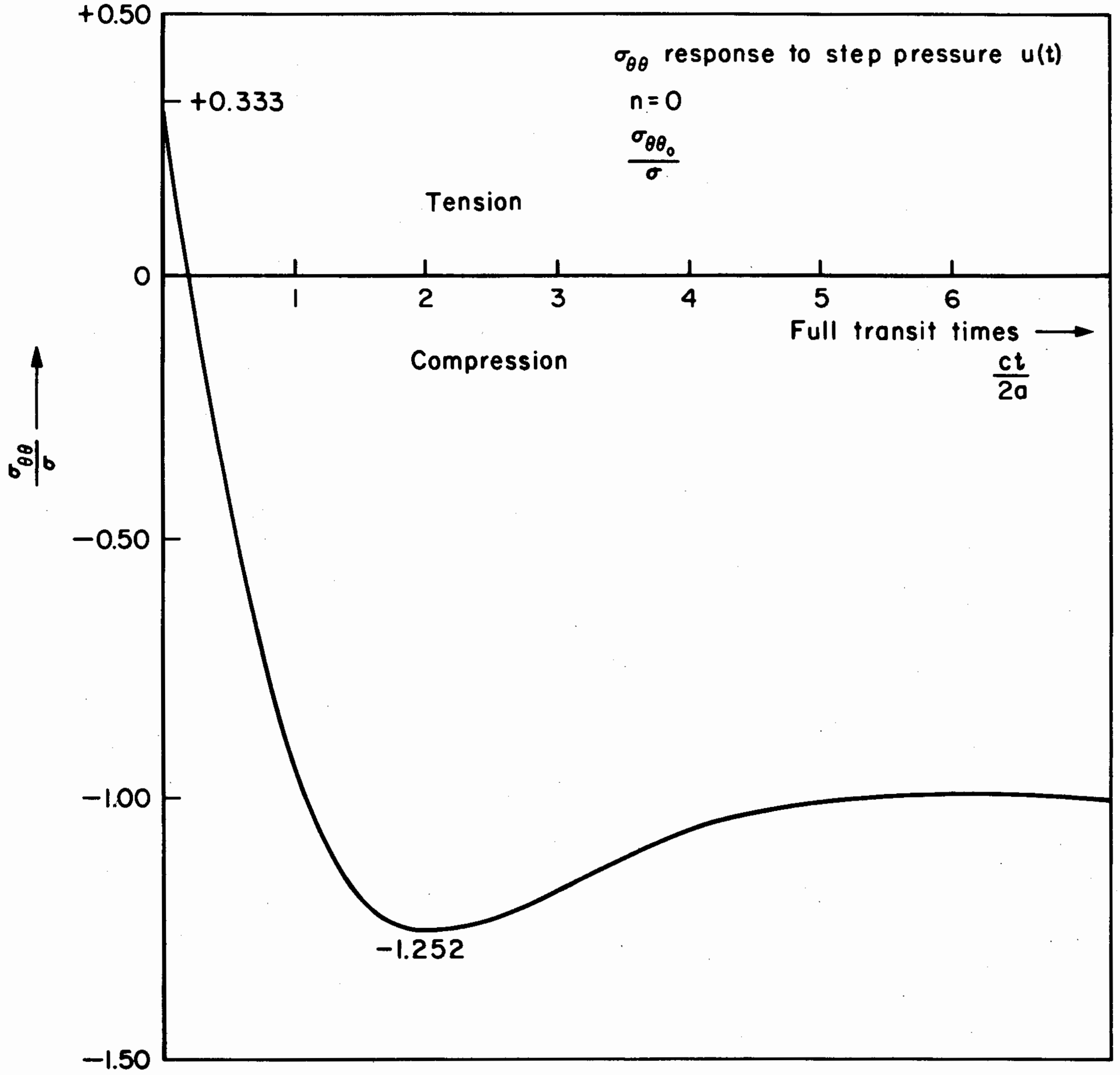


Fig. 7 — $\frac{\sigma_{\theta\theta_0}}{\sigma}$ at r = a due to u(t)

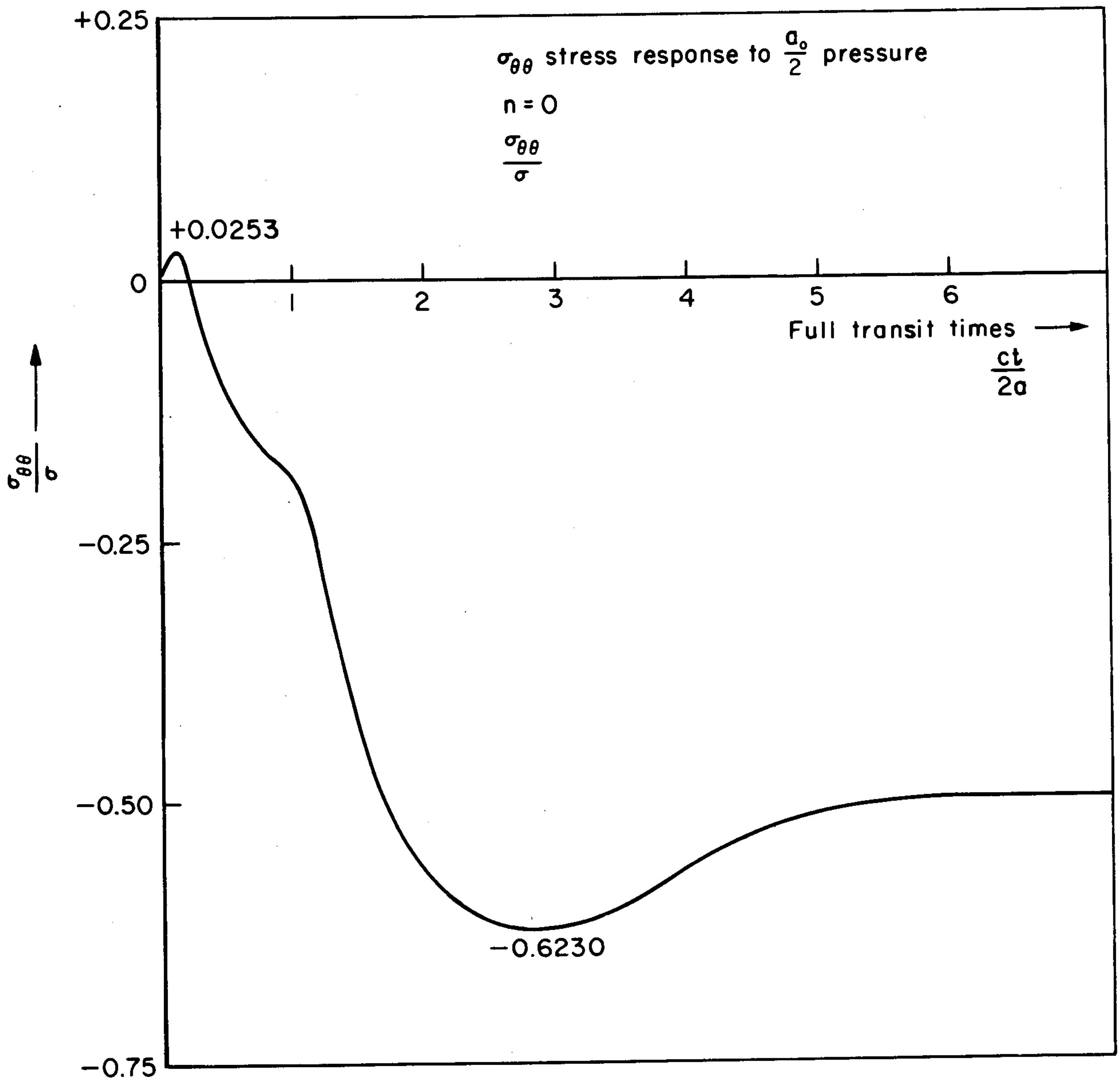


Fig. 8 — $\sigma_{\theta\theta_0}$ at $r = a$ due to $+\ \frac{a_0}{2}(t)$

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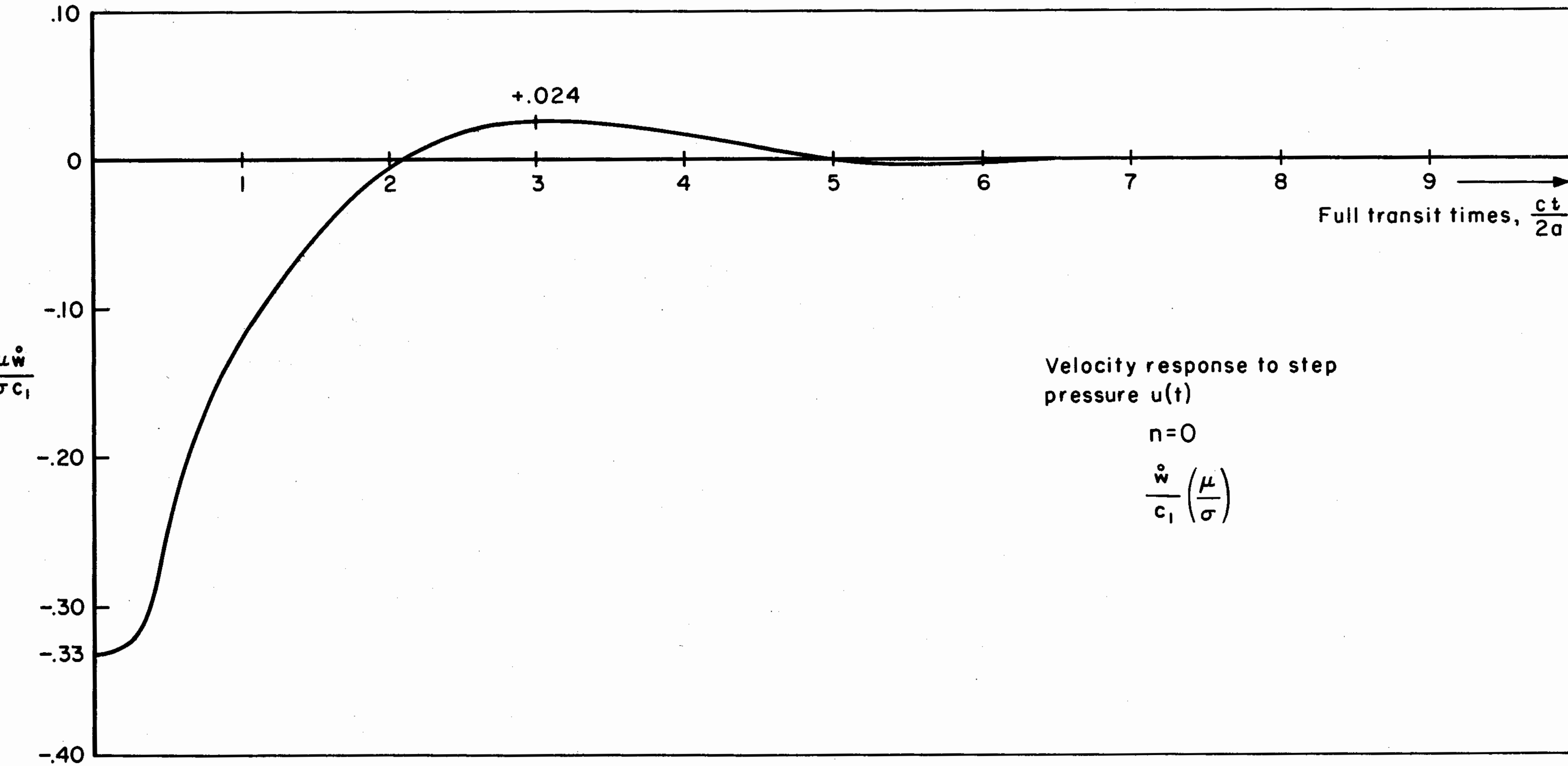

Fig. 9 — Velocity $\mathring{w}(t)$ at $r=a$ due to $u(t)$

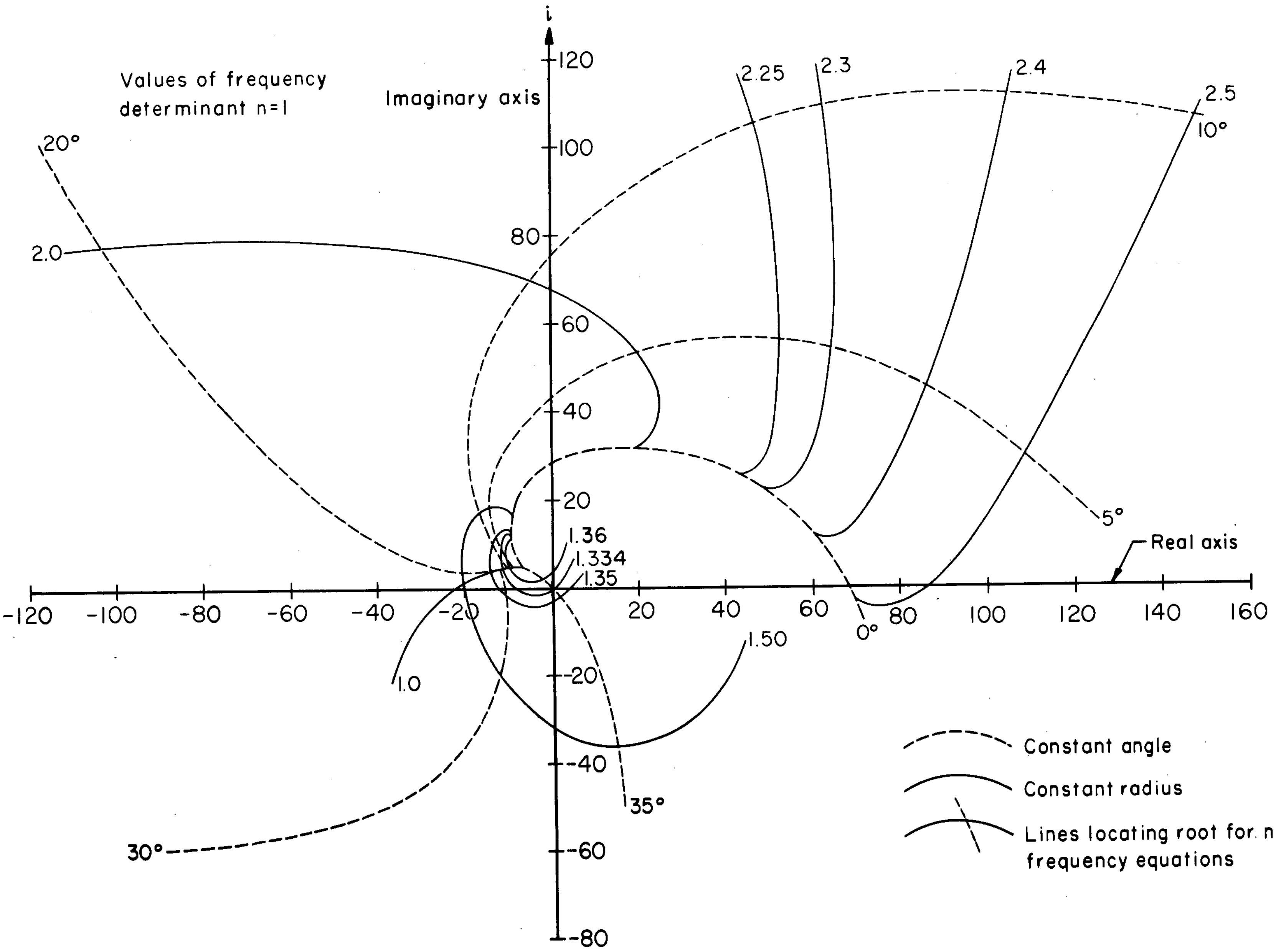

Fig.10—Poles for n=1

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ATTENDEES AT SECOND PROTECTIVE CONSTRUCTION SYMPOSIUM
March 24, 25, 26, 1959

Ackerman, J. O.	Corps of Engineers, United States Army
Adams, Leonard D.	Daniel, Mann, Johnson, & Mendenhall and Associates
Adams, Ray	Anderson-Nichols & Company
Adler, Lawrence	Michigan College of Mining and Technology
Agbabian, M. S.	The Ralph M. Parsons Company
Agnew, George	United States Air Force
Ahliquist, N.	University of California
Albright, Gifford H.	The Pennsylvania State University
Alexander, Anderson B.	United States Air Force
Allen, Colonel H. B.	United States Air Force
Allen, Ralph W.	System Development Corporation
Allgood, Jay R.	United States Naval Civil Engineering Laboratory
Allison, Donald K.	Consultant to Rocketdyne
Alpert, Leo	Corps of Engineers, United States Army
Alsmeyer, William C.	Leo A. Daly Company
Anderson, Donald C.	Armour Research Foundation
Anderson, Douglas R.	The RAND Corporation
Anderson, Harold V.	J. H. Pomeroy & Company, Inc.
Anderson, Major Ralph R., Jr.	United States Air Force
Andrews, Lt. Commander James D.	Armed Forces Special Weapons Project
Angle, Ellwyn E.	Lockheed Aircraft Corporation
Arbuthnot, Guy L., Jr.	Corps of Engineers, United States Army
Armstrong, James W., Jr.	Air Defense Command, United States Air Force

Armstrong, Major Robert H.	Air Force Institute of Technology
Arnovick, George N.	Lockheed Aircraft Corporation
Ash, Milton	System Development Corporation
Attridge, Walter S.	The MITRE Corporation
Bacigalupi, Clifford M.	University of California Lawrence Radiation Laboratory
Baker, Shannon	Lane-Wells Company
Baker, Sherman A.	Office of the Assistant Secretary of Defense (Supply & Logistics)
Balmer, Harold F.	Strategic Air Command, United States Air Force
Baran, Ephraim	Rochlin & Baran
Baran, Paul	Hughes Aircraft Company
Barnett, Cherie	Hughes Aircraft Company
Baron, Melvin L.	Paul Weidlinger Associates
Barrett, J. W.	Guy B. Panero Engineers
Barton, Leonard D.	Canadian Army
Barton, Millard F.	Space Technology Laboratories, Inc.
Bass, Charles D.	The Mosler Safe Company
Beam, Captain John B.	SAC MIKE, United States Air Force
Beatty, Robert L.	Canadian Army
Beck, Christian	United States Atomic Energy Commission
Beck, Earl J.	United States Naval Civil Engineering Laboratory
Beckley, B. G.	The RAND Corporation
Bedrosian, Edward	The RAND Corporation
Beebe, B. K.	Fay, Spofford & Thorndike, Inc.
Beem, Major Samuel M.	North America Air Defense Command, United States Air Force

Bell, Squadron Leader Donald A.	Royal Canadian Air Force
Bennett, R. R.	Space Technology Laboratories, Inc.
Benson, Dana A.	Rome Air Development Center, United States Air Force
Bevan, Roby B.	Palmer and Baker Engineers, Inc.
Biederman, Harry R.	Lockheed Aircraft Corporation
Bishop, A.	Corps of Engineers, United States Army
Bishop, Hoel S., Jr.	Analytic Services, Inc.
Bishop, J. A.	United States Naval Civil Engineering Laboratory
Bjork, Robert L.	The RAND Corporation
Blackman, Captain Herbert	Ballistic Missile Division, United States Air Force
Blacksmith, Philipp, Jr.	Air Force Cambridge Research Center
Blaschke, T. O.	Parsons, Brinckerhoff, Hall & MacDonald
Bleich, Hans H.	Columbia University
Blevans, J. A.	Corps of Engineers, United States Army
Bobisch, W. J.	Bureau of Yards and Docks, United States Navy
Boshkov, Stefan	Columbia University
Bosley, Noble A.	Corps of Engineers, United States Army
Bothun, R. B.	Stanford Research Institute
Bower, John L.	Consultant to The RAND Corporation
Boyer, W. N.	Ballistic Missile Division, United States Air Force
Boynton, William W.	Boynton Associates
Breckenridge, R. A.	United States Naval Civil Engineering Laboratory
Brennan, Lt. Colonel Genald W.	United States Air Force

Breuer, Walter F.	Corps of Engineers, United States Army
Bribiesca, Jose L. Sanchez	Ministry of Hydraulic Resources, Republic of Mexico
Brode, Harold L.	The RAND Corporation
Brooks, John A.	Space Technology Laboratories, Inc.
Brooks, Nancy J.	The RAND Corporation
Brothers, Leroy A.	United States Air Force
Brown, Bernice B.	The RAND Corporation
Brown, R. W.	North American Aviation, Inc.
Brown, Robert E.	Ballistic Missile Division, United States Air Force
Brown, William	The RAND Corporation
Budd, Robert	Walsh Construction Company
Bugg, Sterling L.	United States Naval Civil Engineering Laboratory
Burk, William E., Jr.	Associated Research Design, Inc.
Burkart, William	United States Naval Civil Engineering Laboratory
Burkett, Thomas L.	The Martin Company
Burres, Warren G.	Portland Cement Association
Burton, C. E.	Atlas Copco Pacific, Inc.
Byrne, L. J.	Defence Research Board, Canada
Byrne, W. J.	The MITRE Corporation
Cahn, Albert	The RAND Corporation
Camp, Richard J.	SAC MIKE, United States Air Force
Campbell, Colin A.	Michigan College of Mining and Technology
Cantu, Emilio	Boeing Airplane Company
Cardall, John D., Jr.	Sandberg-Serrell Corporation

Carey, Charles B.	The RAND Corporation
Carlson, R. H.	Sandia Corporation
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Carr, Ralph S., Jr.	Abbott Engineering, Inc.
Carter, Thomas A., Jr.	Beverly Hills, California
Case, John	Case & Gregory
Cash, Ross	Moffatt and Nichol
Cassidy, James P.	Daniel, Mann, Johnson, & Mendenhall & Associates
Cecil, James H., Jr.	International Business Machines Corporation
Chambers, Sam	United States Army
Chao, Chi-Chang	Stanford University
Chapman, Major Harvey J., Jr.	Air Force Institute of Technology
Chauner, Richard C.	Boeing Airplane Company
Cheney, A. Marshall	Planning Research Corporation
Cherbak, Colonel Victor A., Jr.	Air Force Institute of Technology
Chesnut, John D.	Dresser Industries, Inc.
Choate, Lt. Colonel Richard I.	United States Air Force
Chobotov, Vladimir	Space Technology Laboratories, Inc.
Christensen, Commander W. J.	Bureau of Yards and Docks, United States Navy
Christia, A. G.	United States Naval Civil Engineering Laboratory
Churchill, Lt. Colonel Edward	Canadian Army
Ciochetto, F. C.	Corps of Engineers, United States Army
Clark, Robert O.	Ballistic Research Laboratories, United States Army

Clarke, Frank E.	Corps of Engineers, United States Army
Cohen, D.	University of California
Cohen, Edward	Ammann & Whitney
Cohn, D. L.	Space Technology Laboratories, Inc.
Coker, F. B.	United Electrodynamics
Coker, Joseph D.	Office of Civil and Defense Mobilization
Coleman, George W.	Charles Luckman Associates
Conner, William J., Jr.	General Electric Company
Cook, H.	University of California
Cooke, J. Barry	Pacific Gas and Electric Company
Corn, Bob	Planning Research Corporation
Corns, Charles F.	Corps of Engineers, United States Army
Corsbie, Robert L.	United States Atomic Energy Commission
Costanza, Samuel R.	Ballistic Missile Division, United States Air Force
Cox, C. W.	Douglas Aircraft Company, Inc.
Craggs, J. W.	Brown University
Cross, Lt. Colonel Robert D.	United States Atomic Energy Commission
Cross, Commander William H.	United States Navy
Cunny, Robert W.	Corps of Engineers, United States Army
Cushman, Lloyd C.	Northrop Corporation
Daly, D. A.	Kaiser Engineers
Davies, Alan D.	Radio Corporation of America
Day, Max E.	United States Navy
Dean, Major O. G., Jr.	The RAND Corporation
DeJounge, Lars	Sandvik Steel, Inc.
Deptula, Captain Alfred R.	United States Air Force

Dill, Lt. Commander Allen F.	Civil Engineer Corps Officers, United States Naval School
DiMaggio, Frank L.	Columbia University
DiNapoli, P. A.	Ammann & Whitney
Dittemore, M.	Space Technology Laboratories, Inc.
Dohoo, Roy	The RAND Corporation
Dole, Colonel Norman D.	United States Air Force
Donegon, R. E., Jr.	North American Air Defense Command, United States Air Force
Donelson, E. V.,	American Telephone and Telegraph Company
Donovan, R. J.	Burns and Roe, Inc.
Dorman, Kenneth L.	Michigan College of Mining and Technology
Dougan, Colonel Eric	United States Air Force
Douglas, Walter S.	Parsons, Brinckerhoff, Hall & MacDonald
Dowdy, Ross W.	Daniel, Mann, Johnson, & Mendenhall and Associates
Drake, Harry	Corps of Engineers, United States Army
Drucker, Daniel C.	Brown University
Duesterhoeft, W. C., Jr.	Dresser Industries, Inc.
Duke, C. Martin	University of California
Dunbar, Rear Admiral Frank R. (ret.)	Office of Civil Defense, Los Angeles
Duncan, George H.	University of California Lawrence Radiation Laboratory
Duvall, Wilbur I.	United States Bureau of Mines
Dwyer, W. A.	Westinghouse Electric Corporation
Echardt, Walter	Operations Research Office, The Johns Hopkins University
Eckel, Edwin B	United States Geological Survey
Eckert, Colonel C. A.	Air Force Institute of Technology

Egbert, Arthur S.	North American Aviation, Inc.
Eisner, W. D.	The RAND Corporation
Eldridge, Frank R.	The RAND Corporation
Ellington, Lt. Colonel Edward H.	Air Defense Command, United States Air Force
Enright, Paul M.	International Engineering Company, Inc.
Enthoven, A. C.	The RAND Corporation
Ernst, Captain J. C.	United States Air Force
Esling, Robert H.	System Development Corporation
Eubank, William C.	Tactical Air Command, United States Air Force
Evans, Jack	The Martin Company
Facci, Hugo A.	Office of the Assistant Secretary of Defense (Properties and Installations)
Fayman, Edward A.	Lockheed Aircraft Corporation
Fechtman, Carl	The Ralph M. Parsons Company
Flanigan, Roy E.	Air Research and Development Command, United States Air Force
Fleming, Lawrence	Southwestern Industrial Electronics Company
Flowers, Donald	Convair Astronautics
Fontana, Lt. Colonel Robert E.	Air Research and Development Command, United States Air Force
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Freeman, Paul M.	Bechtel Corporation
Fremouw, Colonel Gerrit D.	Supreme Headquarters Allied Powers Europe
Friedmann, L.	Space Technology Laboratories, Inc.
Fry, S.	University of California

Fulmer, Gordon W.	United States Navy
Gardiner, William W.	Office of Civil and Defense Mobilization
Gardner, Maj. General Grandison (ret.)	Associated Research Design, Inc.
Geisinger, Captain Walter F.	Tactical Air Command, United States Air Force
Gelnett, Raymond E.	Corps of Engineers, United States Army
Genensky, Samuel M.	The RAND Corporation
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Gordon, Melvin	Hughes Aircraft Company
Gorgas, Stanley	Goodyear Aircraft Corporation
Gouze, Duane A.	Northrop Corporation
Graham, Fred B.	Tacreon Terminals Company
Graham, Gene	Graham & Klassen Drilling Company, Inc.
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Grenier, Laurent	Foundation of Canada Engineering Corporation Limited

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Grubaugh, Robert E.	E. H. Plesset and Associates
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Hammer, J. G.	University of Illinois
Hanna, J. W.	Convair Astronautics
Hanton, T. F.	Kellogg Switchboard and Supply Company
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Haspert, J. C.	Hughes Tool Company
Hatch, Burton D.	General Electric Company
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Holly, Kenneth E.	Space Technology Laboratories, Inc.
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Hopkins, Henry S.	Boeing Airplane Company
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Hudson, Verne	Boeing Airplane Company
Hung, Francis	North American Aviation, Inc.
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Johnston, Lt. Colonel Robert E.	United States Air Force
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Kortner, Chris H.	Donald R. Warren Company
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Livingston, Clifton W.	Barodynamics, Inc.
Lockwood, Harold K.	Leo A. Daly Company

Loewer, Alvin C.	Keller, Loewer & Associates
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